Appendix G – Coastal Reports and Addendums

TECHNICAL REPORT VOLUME 1
SR520 COASTAL ENGINEERING REPORT
CHANNEL SEDIMENTATION STUDY

This document was prepared by a Professional Engineer.

VLADIMIR SHEPSIS
STATE OF WASHINGTON
34455
REGISTERED PROFESSIONAL ENGINEER

Vladimir Shepsis, Ph.D., P.E.
Principal, Coast and Harbor Engineering

COAST & HARBOR
ENGINEERING
110 Main Street, Suite 103
Edmonds, WA 98020
Ph 425 778.6733
Fax 425 977.7416

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SR520 Coastal Engineering Report – Channel Sedimentation Study

Executive Summary

This report is Volume 1 of a three volume Technical Report prepared by Coast & Harbor Engineering (CHE) under Task Order 1 for the KIEWIT-HNTB SR 520 Pontoon Construction D-B Project and presents the results of work completed under Task 1 of the Scope of Work, Channel Sedimentation Study. Volume 2 of the Technical Report present the results of analysis and numerical modeling conducted for Task 2 of the Scope of Work, Grays Harbor Site Coastal Hydraulic and Geomorphologic Conditions Study. Volume 3 of the Technical Report presents the results and analysis for Task 3, Launch Channel Side Slope Rock Dimensions, and for Task 6, Dolphins Scour Analysis and Protection Design.

A kickoff meeting was conducted between WSDOT and the KIEWIT-HNTB Team on August 10, 2010 to discuss the study approach and methodology. Lacking sufficient measured sediment data for detailed numerical modeling, it was agreed that empirical and analytical sedimentation analysis would be performed. The analysis incorporates actual observed sedimentation rates at nearby sites nearby provide practical sedimentation rate estimates for the launch channel. Supplemental numerical modeling of sedimentation was also performed as a qualitative check of the results from the empirical methods and yielding similar results.

The analysis and estimates of sedimentation rates in the proposed launch channel and recommendations for maintenance dredging requirements were conducted based on compilation and review of existing and historical data, data from prototype projects, and limited qualitative numerical modeling of flow, circulation, and sediment transport. For the purpose of sedimentation rate estimates, the launch channel was divided into two parts. The analysis and predictions of sedimentation where conducted separately for these two parts:

- ***Submerged Part***: Approximately 360 ft long seaward part of the channel where side slopes are typically submerged at low tide.
- ***Emergent Part***: Approximately 250 ft long landward part of the channel where the upper side slopes are emergent at low tide.

Two sites that have similar sedimentation processes were identified as prototype sites and analyzed: Created Slough and the Aberdeen Reach of the Federal Navigation Channel. Created Slough was used as a prototype for the emergent part of the launch channel. Aberdeen Reach was used as a prototype for the submerged part of the launch channel.

- ***Created Slough:*** Data from manmade Created Slough demonstrate that the bottom and slopes have achieved dynamic equilibrium in deposition of suspended sediment and erosion at the bottom. This equilibrium is likely controlled and balanced by small flow velocities in the slough that warrants a limited volume of fresh water and suspended

sediment entering from the Chehalis River, and correspondently minimal re-suspension and erosion of already settled sediment.

- ***Aberdeen Reach:*** Analysis of historical dredging and bathymetric survey data shows that there are two patterns of sedimentation in the federal navigation channel near the project site. The first pattern represents the rapid sedimentation that occurs immediately after dredging. The annual average rate of sedimentation over the channel cross-section from "primary sedimentation" is estimated to be approximately 8.1 ft per year. This primary sedimentation period is estimated to last a maximum six months. The second pattern of deposition represents the period when the dredging cut is partially or completely filled up with sediment, and channel depth appears to have achieved a certain state of dynamic equilibrium. This pattern of deposition is further referenced as "secondary sedimentation." The annual average rate of sedimentation over the channel cross-section from secondary sedimentation is estimated to be approximately 1.8 ft per year.

The sedimentation rate in the emergent part of the channel was estimated based on analysis of the Created Slough prototype project, an empirical method, and review of numerical modeling results. The sedimentation rate in the submerged part of the channel was estimated using multiple methods approach (a total of three). Some input parameters for computations by these methods were obtained from the project prototype. The results (sedimentation rates) obtained by the three different methods were compared, and an estimated design value was derived to provide a somewhat conservative scenario for the proposed channel sedimentation rate.

The recommended sedimentation rate in the submerged part of the launch channel is estimated at 0.72 ft per month during primary sedimentation period and 0.24 ft per month during the secondary sedimentation period. The annual equivalent total thickness of sedimentation in the submerged channel would be approximately 5.8 ft per year. The recommended sedimentation rate in the emergent part of the launch channel is estimated at 0.12 ft per month. The annual equivalent total thickness of sedimentation in the emergent part of the launch channel would be at approximately 1.4 ft per year. In the zone where the submerged channel transitions to the emergent channel near the existing shoreline, the thickness of sediment deposition would also gradually transition. This transition zone would likely range from 50 to 200 ft in length, depending on the sediment properties and nature of sediment consolidation.

The analysis considered a channel bottom elevation of -13.0 ft mean lower low water (MLLW), per the preliminary design drawings. Upon preliminary analysis and in coordination with the Project Team, it was established that the minimum navigable depth in the launch channel is 9.0 ft at MLLW (elevation -9.0 ft MLLW). Because the design channel bottom elevation is below the required navigable elevation, the channel can accommodate 4.0 ft of sedimentation without compromising the navigable depth of the channel. Therefore, this 4.0 ft clearance can be effectively considered as advanced maintenance dredging.

Based on the estimated rates of sedimentation, it appears that the emergent section of the channel would provide the required navigation depth for more than 12 months (one year). Maintenance dredging in this part of the channel may be required approximately once every two or three years, with a volume of approximately 2,200 cubic yards (CY) per year of accumulation.

It appears that the submerged section of the channel would provide the required navigation depth for approximately five to six months. Dredging would then be required to provide the required navigation depth, with a corresponding maintenance volume of approximately 16,000 CY per six months. To reduce the frequency of dredging in the submerged part of the channel, an increase in the advanced maintenance dredging cut elevation is recommended. However, since there is uncertainty in the sedimentation estimates, increasing the advanced maintenance dredging is suggested only after completion of a channel monitoring program following the first six months after initial launch channel construction. The monitoring program (that would include frequent hydrographic surveys) will provide information to ground truth predictions of sedimentation, and help optimize the advanced maintenance dredging cut elevation based on actual experience at the project site.

Note that sedimentation does not occur uniformly in time, but rather varies on scales from hours to months dependent on local conditions. Estimated sedimentation rates and patterns described herein are considered typical average rates and suitable for planning purposes. Monthly rates of sedimentation herein are derived from the average annual rates, and are not intended as precise estimates. Storm events can alter short-term (monthly) and longer-term (annual) rates of sedimentation at the project site.

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Technical Report Volume 1 – DRAFT
SR520 Coastal Engineering Report – Channel Sedimentation Study

1. Introduction

The launch channel would be dredged and excavated to facilitate navigation operations for towing pontoons (SR 520 Bridge Replacement and HOV project) from the casting basin and into the Federal Navigation Channel. The pontoons would then be towed further to a pre-determined moorage location. The project site and launch channel is located at the upper part of the Grays Harbor (as depicted in Figure 1) and would be subjected to sedimentation from suspended and bedload sediment. The objective of the study was to estimate possible rates of sedimentation in the proposed launch channel and develop recommendations for maintenance dredging requirements.

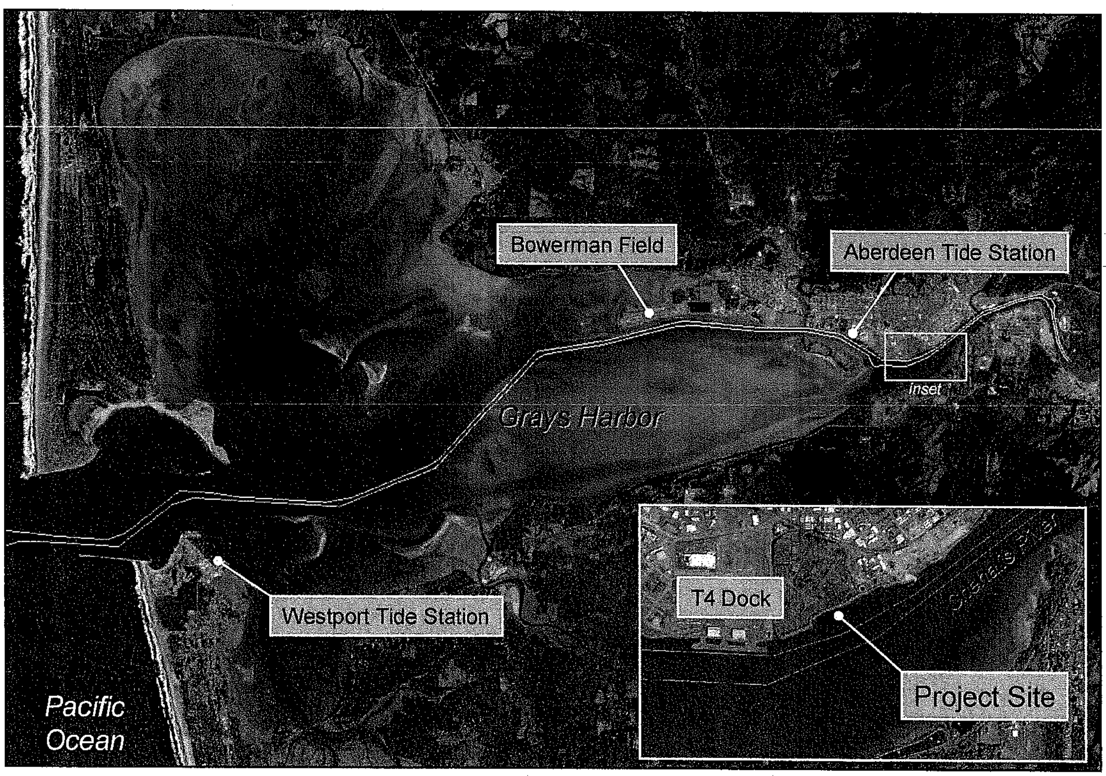

Figure 1. Location map of project site, data stations, and the 2009 background aerial photograph.

The proposed launch channel is designed with a bottom elevation of -13.0 ft MLLW for a width that varies along the channel length as shown in Figure 2. The minimum channel width is 140 ft from toe of slope to toe of slope. Total channel length is approximately 620 ft. As depicted in Figure 3, the channel slopes were originally designed to be armored with rock along the entire channel length. The channel side slopes with armor rock were designed at 1V:3H. The locations of the three cross-sections in Figure 3 are shown in Figure 2. Figures 2 and 3 refer to MLLW, the project vertical datum. Sedimentation analysis conducted herein was performed for fully armored side slopes at 1V:3H, as depicted in Figures 2 and 3.

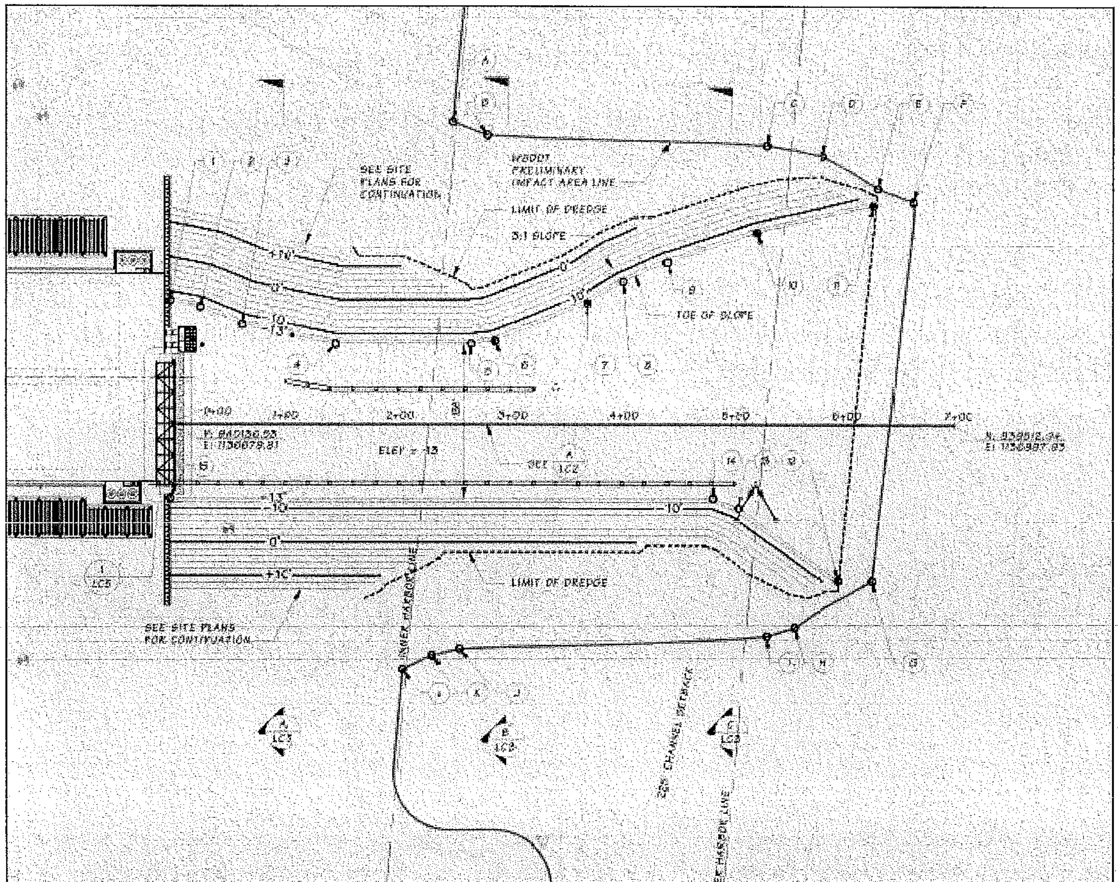

Figure 2. Plan view of the proposed launch channel (drawing provided by HNTB)

Following preliminary analysis, CHE suggested that the Project Team evaluate a means to minimize the extent of armoring by locating armor rock only along the landward-most 200 ft of channel length. The remaining part of the channel (primarily the submerged channel) could then be left unarmored; thus, maintaining the channel at this area with a natural slope of repose of approximately 1V:5H (see geotechnical report). Upon approval by the Project Team (meeting on August 31, 2010) the preliminary design cross-section with no armoring of the slope along the submerged part of the channel was performed. If selected for detailed design, sedimentation and maintenance dredging requirements for this partially armored channel condition will require further analysis.

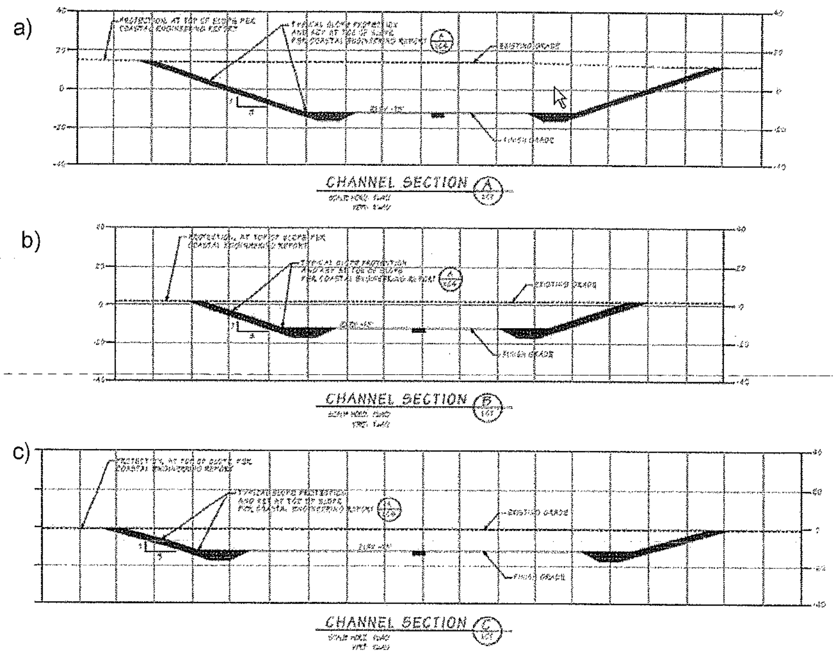

Figure 3. Cross-sections of original launch channel design, (a) landward, (b) middle, (c) seaward parts (drawing provided by HNTB)

Physical conditions and factors controlling sedimentation in the launch channel vary along the channel length. For example, the launch channel sideslopes from the most seaward Station 6+10 to approximately the Inner Harbor line at Station 2+30 are submerged during most tidal conditions (excluding extreme low tides). The launch channel side slopes from approximately the Inner Harbor line at Station 2+30 landward to the casting basin gate at Station 0+00 are frequently above the tide (excluding extreme high tide events).

Sedimentation in the submerged channel (Station 2+30 to 6+10)would depend on waves and cross-channel current fluxes, re-suspension of adjacent bottom slopes upstream and downstream from the channel, and formation of mud gravity flows on the channel slopes. However, these factors may not be present at all or may be unimportant contributors to sedimentation in the emergent part of the channel. Therefore, for more accurate estimates and to account for different processes, the analysis of sedimentation was conducted separately for the submerged (seaward) and emergent (landward) channel.

In the zone where the submerged channel transitions to the emergent channel near the existing shoreline at Station 2+30, the thickness of sediment deposition would also gradually transition. This transition zone would likely range from 50 to 200 ft in length, depending on the sediment properties and nature of sediment consolidation.

The sedimentation rate in the emergent part of the channel (Station 0+00 to 2+30) was estimated based on analysis of a prototype project upstream on the Chehalis River, review of numerical modeling results, and an empirical approach. The sedimentation rate in the submerged part of the channel was estimated using multiple (a total of three) methods. Some input parameters for computations by these methods were obtained from a project prototype. The resulting sedimentation rates obtained by different methods were compared and an estimated design value was derived, accounting for conservatism in the proposed channel sedimentation rate. Sedimentation in the transition zone was linearly interpolated between submerged and emergent sedimentation patterns by using an angle of repose derived in part from the existing mudflat geometry.

2. Channel Prototypes Selection and Data

Two prototypes of the launch channel were identified and analyzed: (1) Created Slough and (2) Cow Point/Aberdeen Reach of the Federal Navigation Channel.

2.1. Prototype 1: Created Slough on the Chehalis River

The Created Slough prototype was designed and constructed 4 miles upstream of the project site on the lower Chehalis River by the U.S. Army Corps of Engineers-Seattle District as part of the Grays Harbor Navigation Improvement Project (GHNIP) in 1991. The location and a recent oblique aerial photograph of the Created Slough are shown in Figure 4.

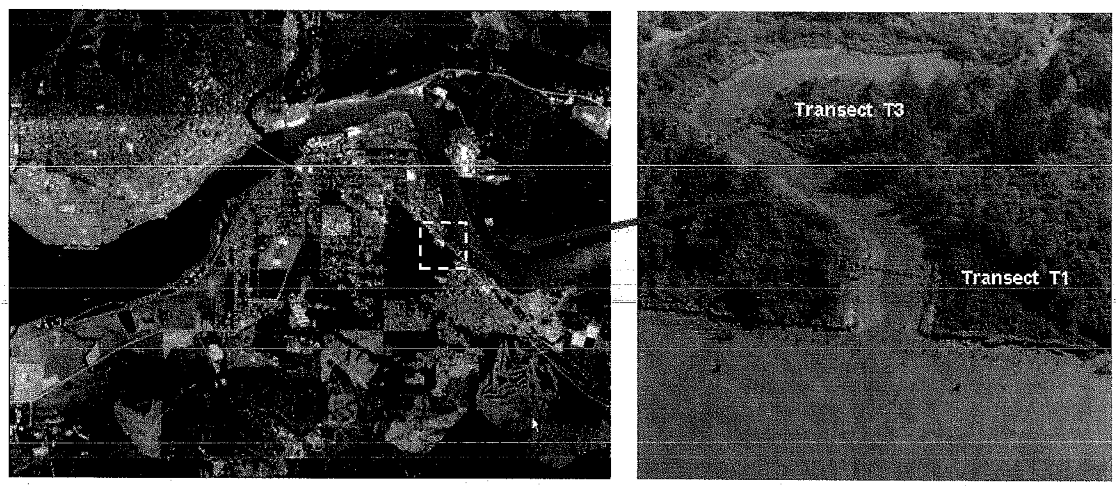

Figure 4. Created Slough prototype (a) location and (b) oblique aerial photograph from 2002.

The slough was constructed approximately 1200 ft long with a width varying from 120 ft to 150 ft. Based on limited information, the design elevation of the slough channel bottom was approximately -4 ft MLLW. However, the as-built drawings are not available and the actual depth of the constructed slough is not known.

Two monitoring topo-bathymetric surveys were conducted by the University of Washington at four survey transects along the slough in 1995 and 2000. The survey transects were spaced at approximately equal distances along the slough. The most seaward transect, T1 (closest to the mouth in Figure 4 above) is located approximately 150 ft inland from the entrance to the Chehalis River[1]. Transect T3, shown above in Figure 4, is approximately 600 ft inland from the Chehalis River.

Plotted survey data at transects T1 and T3 are shown in Figure 5 (courtesy of Simenstad 2001). The figure shows a change in elevation across the slough at the two transects for the period 1995-2000. Red color in the figure shows a design Created Slough cross-section[2].

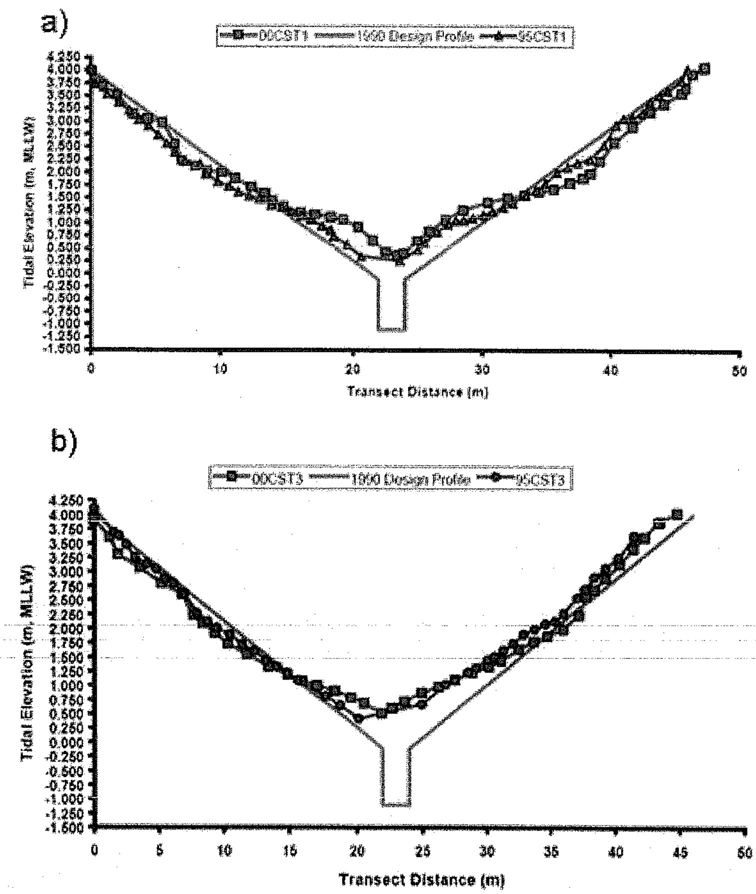

Figure 5. Monitoring surveys from 2000 (blue squares) and 1995 (purple shapes) superimposed at Transects T1 (a) and T3 (b). Note that units are in meters.

The data in the figure show no significant change in slough cross-sectional configuration at transect T1 and further inland during the five-year period. The maximum change in elevation for the five-year period did not exceed 2 ft[3]. It appears that the slough cross-sections have achieved a certain dynamic equilibrium between

[1] Numbering of transect increases with distance from the Chehalis River.
[2] The quantity estimate of sedimentation rate during the post-construction period may not be reliable because of absence of post-construction survey data and gaps in survey data between 1991 and 1995.
[3] These changes may also be attributed to the accuracy of the survey and plotting in the figure.

deposition of suspended sediment and erosion of surface sediment due to tidal-river flow. This equilibrium has likely been controlled and balanced by small flow velocities in the slough that warranted a limited volume of water and suspended sediment entering from the Chehalis River, and minimal re-suspension and erosion of already settled sediment. Similar effects of low velocities, small volume of suspended sediment, and small deposition is expected at the embedded part of the launch channel. This similarity is accounted for in estimating the sedimentation rate for the launch channel (See Section 3).

2.2. Prototype 2: Aberdeen Reach of the Federal Navigation Channel

The Aberdeen Reach of the Federal Navigation Channel is located between Stations 1251+87 and 1315+86 and has a total length of 6,399 ft. Aberdeen Reach is directly adjacent to the launch channel that warrants similarity between the prototype and project site conditions relative to waves, flow velocities, and sediment transport. The channel at this reach is maintained at navigable depth of 32 ft MLLW and width of 200 ft. Approximate downstream boundary of the Aberdeen Reach, location of the proposed project, and depth conditions upper Grays Harbor are shown on Figure 6. The Cow Point reach is located downstream of the Aberdeen Reach.

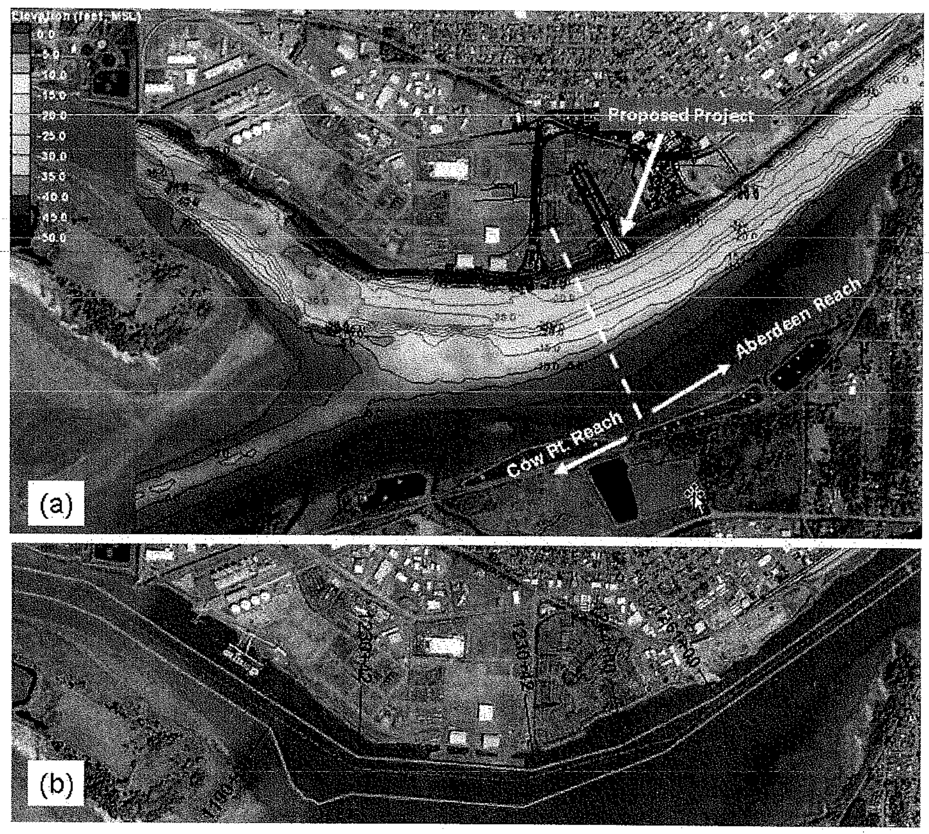

Figure 6. Approximate (a) boundary of the Cow Pt. and Aberdeen Reach, location of the proposed project, and depth conditions (ft Mean Sea Level) in the area (b) Federal Navigation Channel stationing.

Regular maintenance dredging has been conducted by the Seattle District Corps of Engineers (COE) in Aberdeen Reach and adjacent Cow Pt. Reach to provide navigable depth for safe navigation of the design vessels. Dredging records at Aberdeen Reach and Cow Pt. Reach were compiled and analyzed to evaluate historical maintenance dredging volumes in the prototype channel[4]. Table 1 depicts the frequency and volumes of maintenance dredging at these reaches.

Table 1. Historical Volume of Maintenance Dredging at Cow Pt. and Aberdeen Reach from 1997 to 2006 (from HDR 2009).

Year	Between Stations		Volume Dredged (CY)
1997	1167+50	1228+00	310,025
1997	1165+00	1245+50	366,259
1998	1195+50	1229+49	178,302
1998	1246+40	1251+87	6,317
1998	1215+50	1252+00	157,375
1998	1161+00	1228+25	560,417
1999	1156+02	1251+87	417,058
1999	1251+87	1353+56	54,270
1999	1156+02	1251+87	279,987
2000	1156+02	1251+87	203,352
2000	1156+03	1251+88	240,167
2001	1168+80	1233+50	271,303
2001	1164+50	1251+65	380,110
2002	1168+10	1252+00	325,004
2002	1164+50	1251+65	169,482
2002	1164+51	1251+66	211,316
2003	1194+00	1257+00	168,228
2003	1163+40	1231+55	587,567
2004	1156+02	1251+65	197,383
2004	1163+40	1231+55	484,637
2005	1156+02	1251+65	173,352
2005	1161+82	1251+96	511,295
2006	1193+67	1264+80	127,049

The table shows volumes of dredging in Cow Pt. Reach and Aberdeen Reach from approximately Station 1156+00 to Station 1354+00. The estimated average yearly

[4] Please note that the predominant dredging work was conducted along Cow Pt. to provide navigable depths at the Port of Grays Harbor Terminal 4. Using the Cow Pt. dredging data to characterize Aberdeen Reach maintenance dredging requirements is a conservative approach (volumes of dredging are over-estimated for the Aberdeen Reach).

dredged volume at this part of the channel is approximately 725,000 CY per year, or approximately 36 CY per linear foot of the channel per year.

It should be noted that the dredging volumes mostly reflect the COE's budgetary and environmental constraints, and do not reflect the actual rate of sedimentation. In order to estimate actual sedimentation rates in the channel, historical bathymetric survey data were compiled and analyzed.

Historical bathymetric survey data covering Cow Pt Reach and Aberdeen Reach areas were compiled from COE's database for the period from 2000 to the present time. Sequential post-dredge, pre-dredge, and channel conditions survey data were evaluated for analysis of sedimentation. These survey data were coupled to assure that no dredging events occurred between the dates of the survey. A total of 23 datasets of pre- and post-dredge surveys were identified. Sequential bathymetric survey data were compared and depth differences were plotted to derive the sedimentation rate and patterns.

Figure 7 is an example of bathymetric survey difference plots for six periods of time when no dredging work occurred between surveys. The figure includes the periods 02/06/02-10/04/02 (approximately 8-month period); 02/07/03-10/16/03 (approximately 8-month period); 02/02/04-09/27/04 (approximately 8-month period); 02/18/05-06/29/05 (approximately 4-month period); 12/28/05-01/23/06 (approximately 1-month period), and 01/03/07-04/08/08 (approximately 13-month period).

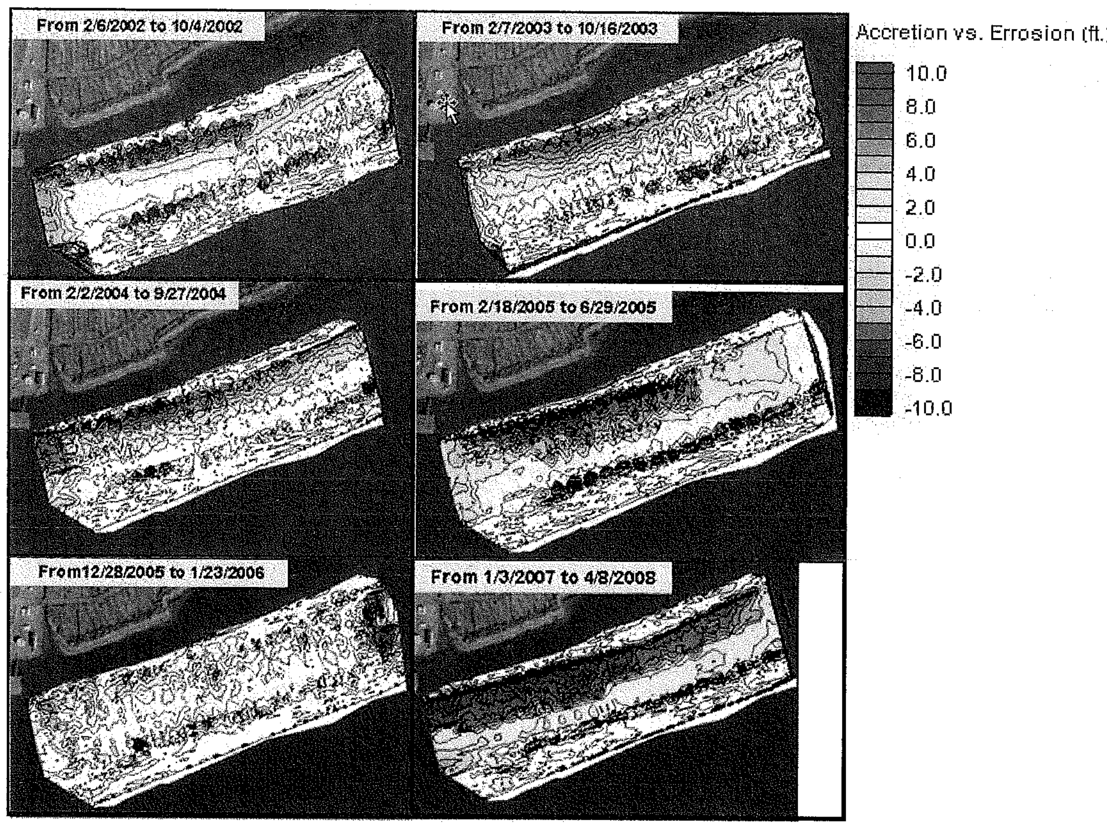

Figure 7. Example of bathymetric survey difference plots for the periods indicated (red indicates accretion and blue indicates erosion)

Figure 7 shows, in color format, the change of water depths in part of the federal navigation channel that is in closest proximity to the project site (red indicates accretion and blue indicates erosion). The figure shows that the rate of sedimentation varies significantly from one period to another. The maximum adjusted annual sedimentation rate varies at different periods and different locations from about 6 ft to14 ft per year. The cross-section averaged annual sedimentation rate varies from approximately 1 ft to 4 ft per year. However, the pattern of sedimentation is relatively similar for all observation periods. This pattern is characterized by a higher rate of accretion along the northern side of the channel, and a lower rate (including some erosion) on the southern side of the channel.

Additional information on pattern and rates of sedimentation was obtained from analysis of two channel cross-sections with plotted sequential survey data. Figure 8 shows the location of selected cross-sections. Section 1 is located downstream and Section 2 upstream from the project site. Figure 9 and Figure 10 plot sequential survey data after the dredging event in January 2007 through October 2009. Dredging did not take place after January 2007 at these locations.

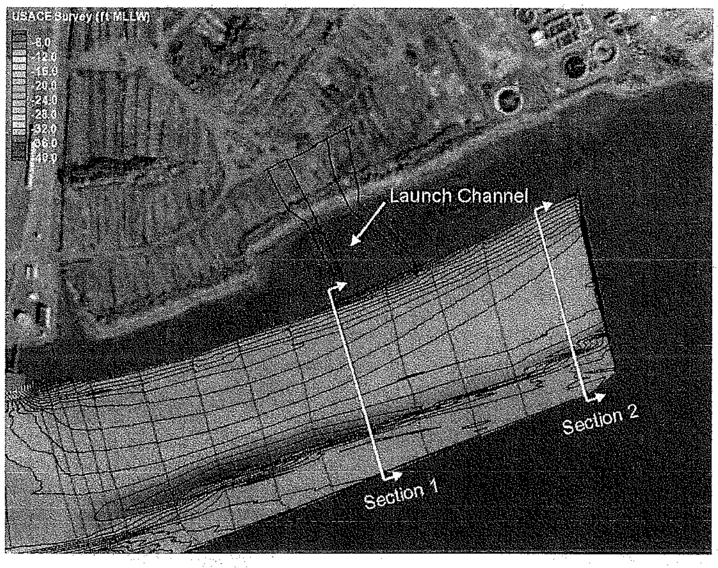

Figure 8. Locations of selected cross-sections and October 2009 bathymetry data

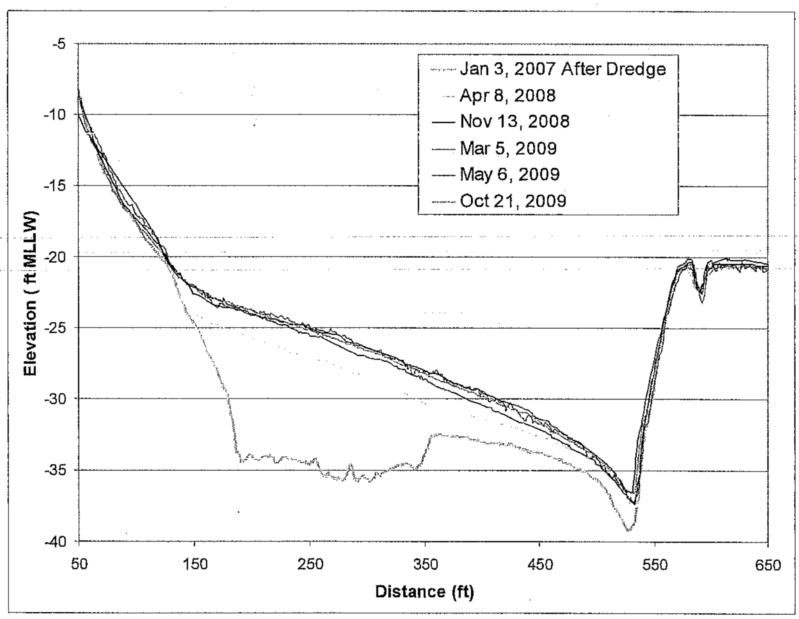

Figure 9. Cross-Section 1 from Jan. 2007 through Oct. 2009

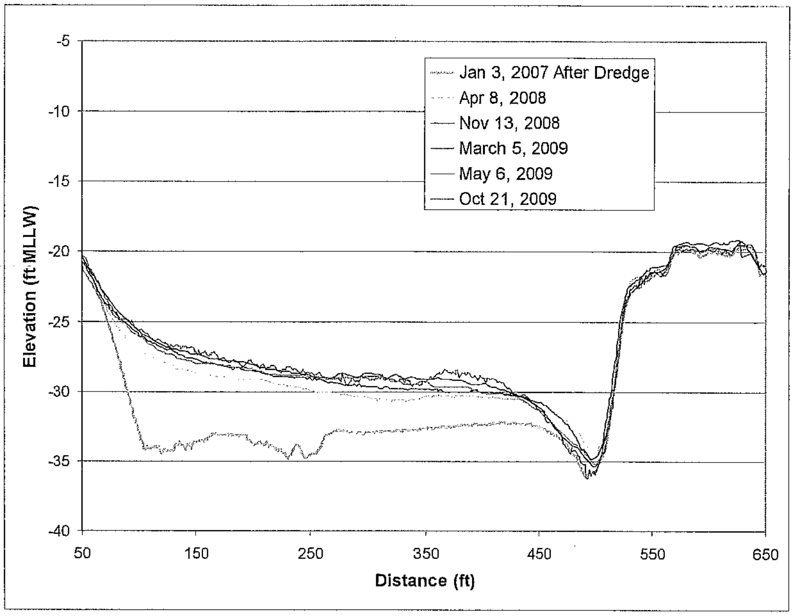

Figure 10. Cross-Section 2 from Jan. 2007 through Oct. 2009

The figure shows (green color) bottom depth from a post-dredge survey of January 3, 2007 and changes of these depths from subsequent surveys on April 8, 2008 (3 months), November 13, 2008 (10 months), March 5, 2009 (14 months), May 6 2009 (16 months), and October 21, 2009 (22 months). Based on the analysis, it appears that there are two patterns of sedimentation in the channel. One pattern represents rapid sedimentation that occurred immediately after dredging. During this "primary sedimentation" period, the rate of deposition is higher and distribution of sedimentation over the channel width is not uniform. The highest deposition for this pattern is observed at the deepest part of the dredging cut along the north side of the channel.

A second pattern of deposition corresponds to a period when the dredging cut is partially or completely filled up, and channel depth appears to have achieved a certain state of dynamic equilibrium. This pattern of deposition is further referenced as "secondary sedimentation." The rate of sedimentation during this dynamic equilibrium period is lower, and distribution of deposition over the channel width is more uniform.

The cross-section averaged rate of sedimentation during the primary sedimentation period, adjusted to an annual rate, is estimated to be approximately 9.0 ft and 7.2 ft for Section 1 and Section 2, respectively. For further analysis the average rate of these two locations, 8.1 ft per year (0.68 ft/mo), is used to describe a primary period of sedimentation. The primary period of sedimentation may vary significantly from year to year and depends on the dredging depth. For simplicity of analysis, it is assumed that the duration of primary sediment deposition is equal to approximately six months.

An average over the channel width (cross-section) rate of secondary sedimentation is approximately 2.4 ft and 1.2 ft per year for Section 1 and Section 2, respectively, which averages approximately 1.8 ft per year (0.15 ft/mo). These rates of sedimentation are used further as prototype sedimentation rates for estimates of launch channel (submerged part) sedimentation rates.

3. Submerged Channel Sedimentation Estimates

The prediction of sedimentation for the submerged part of the launch channel was conducted with three different methods; two empirical and one theoretical. The two empirical methods included: Method 1- U.S. Army Corps of Engineers "Rule of Thumb" and Method 2- Advanced Maintenance Dredging Volume Assessment (AMDVA) Method. The theoretical method included numerical modeling of sediment transport using a 2-Dimensional hydrodynamic model MORPHO. Computations of sedimentation rates with empirical methods were computed for two conditions: the launch channel immediately after dredging (primary sedimentation), and secondary sedimentation in the launch channel. Qualitative computation of sedimentation rates with numerical modeling was conducted for secondary sedimentation only (See Section 3.3).

3.1. Empirical Method 1: Sedimentation Rate Estimates

The "Rule of Thumb" method is based on the following empirical formulation:

$$I = \left(\frac{d_0}{d_n}\right)^m \qquad (1)$$

Where:
I = coefficient of infill
d_0 = dredged channel depth, feet
d_n = natural depth - average along channel centerline depth that existed prior to channel dredging, feet
m = empirical coefficient

The coefficient of infill represents a rate of reduction in channel depth due to sedimentation. A coefficient of infill can be expressed through the channel dredging cut and sedimentation rate as follows:

$$h = d_c(I-1)/I \qquad (2)$$

Where:
h = rate of sedimentation, feet/year
d_c = dredged cut depth (d_0 - d_n)

Substituting formula (1) into formula (2) results in the following:

$$h = d_c\left[1-\left(\frac{d_n}{d_0}\right)^m\right] \qquad (3)$$

The value of empirical coefficient "m" was obtained based on the prototype data for the Federal Navigation Channel, Aberdeen Reach (See Section 2). The Aberdeen Reach primary sedimentation rate (after dredging) was estimated at 8.1 ft per year[5]. The prototype secondary rate of sedimentation was estimated at 1.8 ft per year for the Aberdeen Reach. In computations of empirical coefficient and rates of sedimentation a natural depth (d_n) for Aberdeen Reach was estimated at 17 ft and natural depth for the launch channel was estimated at 2 ft. Using Method 1, the rates of sedimentation in the launch channel were estimated at 8.8 ft ft per year (or 0.73 per month) for primary sedimentation and 5.9 ft per year (or 0.49 ft per month) for the secondary sedimentation.

3.2. Empirical Method 2: Sedimentation Rate Estimate

The AMDVA method (Shepsis and Demich 1993) is an empirical method that estimates the volume of required maintenance dredging in submerged navigation channels, subjected to wave impact. A sedimentation rate by this method is derived from the following equation:

$$h = a * d_0 * H / \sqrt{d_c} \qquad (4)$$

Where:
h = annual sedimentation rate, feet/year
d_0 = channel depth measured, feet
H = maximum yearly significant wave height at channel seaward end, feet
d_c = natural depth, feet
a = empirical coefficient

The value of empirical coefficient "a" was obtained based on the data from the Aberdeen Reach prototype project of the Federal Navigation Channel. The rates of sedimentation at the prototype project were estimated to be 8.1 ft per year and 1.8 ft year per year for primary and secondary sedimentation, respectively. Similar to Method 1, the natural depth (d_n) for Aberdeen Reach was input equal to 17 ft. The rates of sedimentation in the submerged part of the launch by Method 2 were estimated at 8.5 ft per year (or 0.71 ft per month) for the primary sedimentation and 1.9 ft per year (or 0.16 ft/month) for secondary sedimentation, respectively.

3.3. Method 3: Numerical Modeling Method Sedimentation Rate Estimate

Qualitative numerical modeling of channel sedimentation was performed using a 2-Dimensional MORPHO numerical modeling system. MORPHO simulates tidal and river flow hydrodynamics, sediment transport, and computes resulting bottom depth (morphology) changes (Kivva *et al.* 2004). The hydrodynamics and sediment transport/morphology are dynamically coupled, allowing morphological acceleration

[5] For conservatism, the rates estimated for Section 1, downstream from the project site are used herein.

and simulation of long periods of time. Launch channel sedimentation modeling was conducted on a small nested modeling domain, shown in Figure 11.

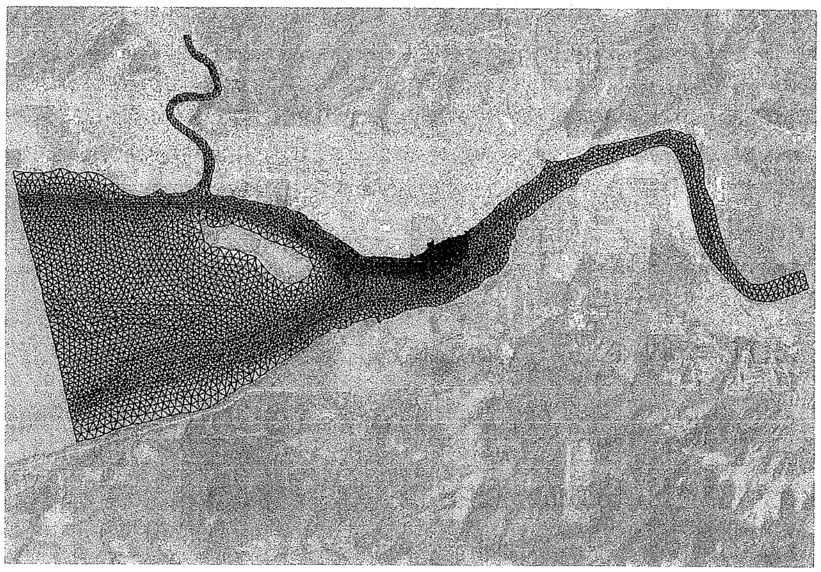

Figure 11. Nested domain for MORPHO sedimentation modeling

The nested MORPHO modeling domain was forced by the large Grays Harbor-wide SELFE hydrodynamic model (Please see CHE Coastal Engineering Report Volume 2). Boundary conditions for the nested model were tidal elevations on the seaward boundary and Chehalis River flow on the river boundary. Flow in the Chehalis River was constant during the modeling period.

Initially, the MORPHO model simulations were conducted to reproduce observed sedimentation patterns and rates in the project prototype, the Aberdeen Reach. For model comparison, the period from November 13, 2008 to March 5, 2009 (0.31 years) and corresponding hydrographic survey data were selected. This period corresponds to secondary sedimentation in the prototype channel[6] (See Section 2).

The bottom depth changes based on actual survey data in the Federal Navigation Channel during this period are shown in Figure 12. The pattern of sedimentation during this period was relatively uniform over the channel area, with estimated average over the area at approximately 1.7 ft. A MORPHO model calibration was conducted by altering sediment transport parameters and input flow rate to achieve similarity in pattern and rate of sedimentation[7].

[6] Despite many attempts, a satisfactory result of numerical model calibration has not been obtained for primary sedimentation conditions (See Section 2 of the report).

[7] Input sediment concentration in the Chehalis River during the modeling period was held constant.

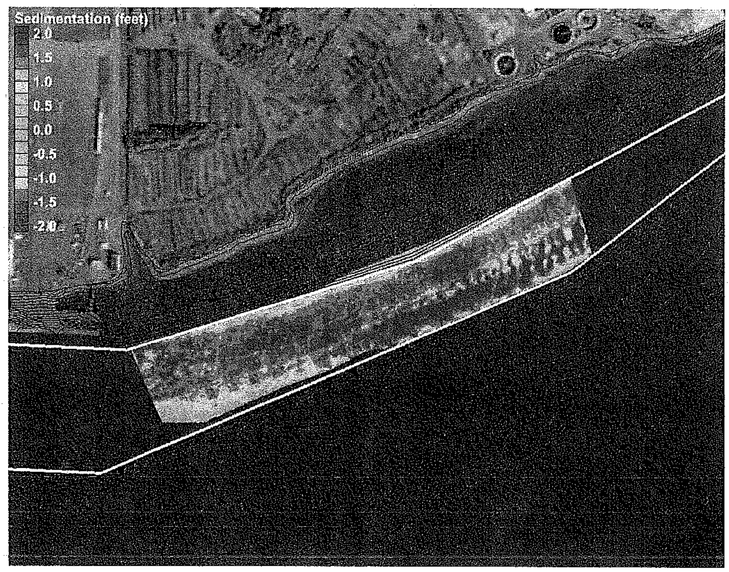

Figure 12. Measured sedimentation in the Federal Navigation Channel during the period November 13, 2008 to March 5, 2009 (0.31 years)

Calibration (by alteration of the model parameters) was conducted until sedimentation in the Federal Navigation Channel matched the sedimentation observed based on hydrographic survey data. A best fit was obtained for the river flow corresponded to 500 m^3/sec (17,700 cfs). This flow is typical of winter flow rates and actually corresponds to a Chehalis River flow measured during early January 2009. Results of calibration are shown in Figures 13 and 14. Figure 13 compares the modeling in plan view in the form of measured (left) and predicted (right) sedimentation thickness during the modeling period of 0.31 year. Figure 14 compares modeling and measured results along the typical transect across the federal channel.

Analysis of the figures shows that, in general, the thickness and pattern of sedimentation in Aberdeen Reach were qualitatively reproduced by the numerical model. Maximum values of sedimentation in the channel are consistent between the observations and modeling results. Considering the complexity of natural processes and uncertainties with input parameters, the calibration of the model is considered to be sufficient to qualitatively simulate sedimentation in the channel for secondary sedimentation conditions.

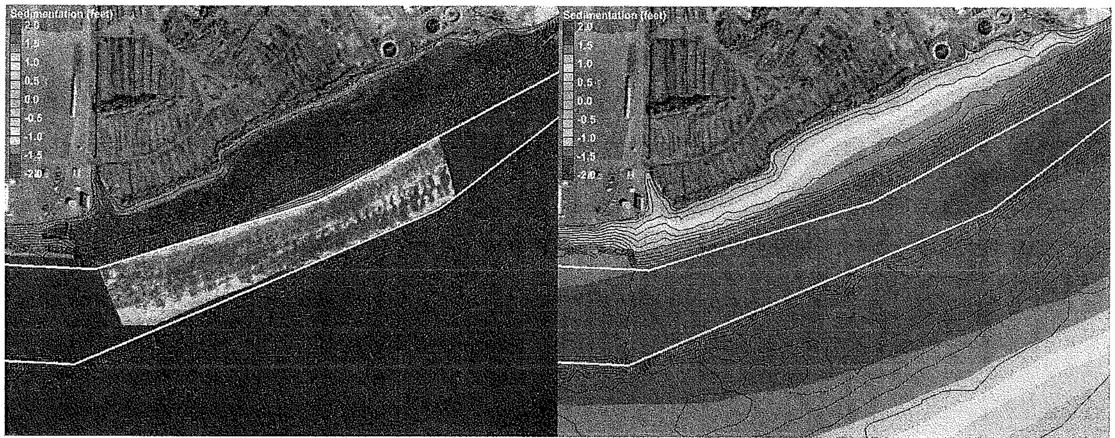

Figure 13. Results of model calibration in plan view: measured sedimentation (left) and sedimentation predicted by the MORPHO model during the period November 13, 2008 to March 5, 2009 (0.31 year). Color scales are identical.

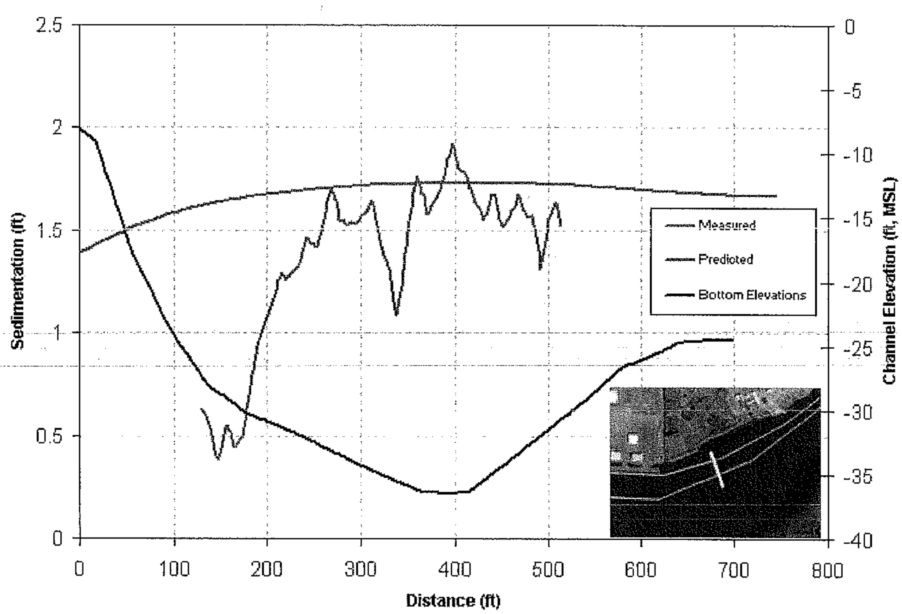

Figure 14. Results of model calibration in cross sectional view

Once calibrated, the MORPHO model was used to conduct analysis of secondary sedimentation in the proposed launch channel. The launch channel was built into the modeling domain and numerical modeling was repeated using similar input parameters as for calibration effort. Results of the modeling, thickness of sedimentation in the launch channel projected for a one-year period, are shown in Figure 15.

The results indicate that the maximum sedimentation would occur in the seaward part of a submerged part of the channel. The sedimentation rate reduces towards the landward part of the submerged channel, and is small near the emergent channel[8]. The averaged sedimentation rate over the submerged channel area is estimated to be approximately 2.9 ft per year (or 0.17 ft/month).

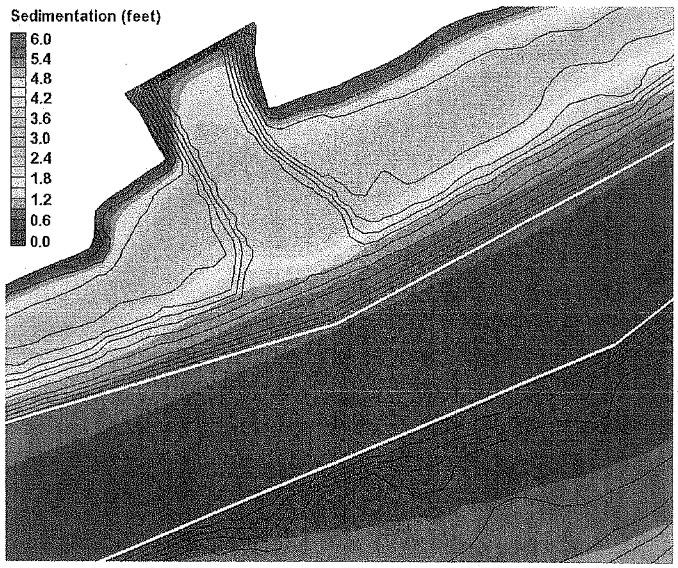

Figure 15. MORPHO modeling results: projected to one year secondary sedimentation rate at launch channel

3.4. Submerged Channel Sedimentation Summary

Three different methods of analysis were used for prediction of sedimentation in the submerged part of the proposed launch channel. The results of estimates by these three methods are summarized in Table 2. Method 3 refers to the numerical modeling results. The table also includes the observed averaged sedimentation rates at the prototype channel, Aberdeen Reach, for reference.

Table 2. Summary of sedimentation rate estimates in the submerged channel with three different methods

Period of Sedimentation	Method 1 (Empirical) ft/month	Method 2 (Empirical) ft/month	Method 3 (Numerical) ft/month	Prototype 2 (Aberdeen Reach) ft/month	Recommended ft/month
Primary	0.73	0.71	-------	0.68	0.72
Secondary	0.49	0.16	0.17	0.15	0.24

The recommended design value of sedimentation for the submerged part of the channel is obtained by averaging results obtained by Methods 1 and 2 for the primary

[8] The modeling results correspond well to a pattern of sedimentation that was observed at Prototype 1, Created Slough. Insignificant sedimentation was observed at the landward part of the slough. This result is used further to derive sedimentation rates at the emergent part of the channel in Section 4 below.

sedimentation and Methods 1, 2, and 3 and the prototype data for the secondary sedimentation.

4. Emergent Channel Sedimentation Estimates

Sedimentation in the emergent part of the channel would occur predominately due to settling (deposition) of suspended sediment from a water column inside of the emergent part of the channel. Some sedimentation may also occur by gravity flows of sediment from the submerged part of the channel, where thickness of deposition would be higher than in the emergent channel. This type of sedimentation would occur at the transition zone and is accounted for by linear interpolation.

In order to estimate the rate of sedimentation from deposition of suspended sediment, a minimum two parameters are required: (1) the volume of water that circulates through the emergent channel, and (2) suspended sediment concentration in this circulating water. The volume of circulated water in the emergent channel was estimated based on flow hydrodynamic modeling (for details see Volume 2 of this report) In the absence of suspended sediment concentration data in the lower Chehalis River, the Calawah River was selected as a prototype, and suspended sediment concentration was derived from measured data on the Calawah.

4.1. Volume of Circulated Water

Flow circulation numerical modeling demonstrated that flow velocities inside of the emergent channel are very low and result in a small volume of water exchange between the launch channel and the federal navigation channel during a single tidal cycle. Figure 16 is an example of numerical modeling, and shows a snap shot of distribution flow velocities in the launch channel during peak ebb flow. Blue colors indicate low velocities (less than 0.5 knot).

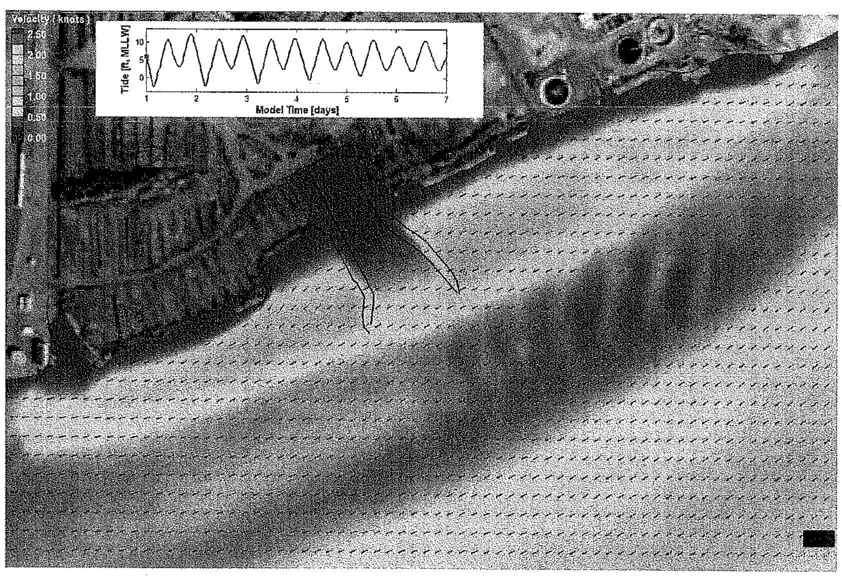

Figure 16. Example of flow circulation numerical modeling

From the figure and numerical modeling, velocities in the emergent channel are small and do not often exceed 0.1 to 0.2 ft/sec. Small current velocities and the expected pattern of circulation would limit water exchange between the emergent channel and federal navigation channel during each tidal cycle. For simplicity of analysis, it is assumed that a complete water exchange in an emergent part of the launch channel would occur during every flood and ebb cycle. In other words, the resident time for the emergent part of the launch channel is equal to approximately 6 hours. Based on this assumption, a water volume that circulates through the emergent channel during each flood and ebb cycle is equal to the channel geometry bounded by horizontal plane at an elevation corresponding to the high tide level. For simplicity, water elevation is assumed to be Mean Higher High Water (MHHW).

Suspended Sediment Concentration: While turbidity measurements are available, no suitable measured suspended sediment concentration data for the Chehalis River and upper part of Grays Harbor have been identified. For the purpose of launch channel sedimentation rate estimates, suspended sediment concentration data from a river-analog was used. Calawah River USGS station 12043000, located on the west side of the Olympic Peninsula, was used as a river-prototype for extracting and extrapolating suspended sediment concentration data for the Chehalis River. A statistical empirical relationship between flow discharge and suspended sediment concentration at Calawah River was plotted (L. Nelson 1982), and is shown in Figure 17.

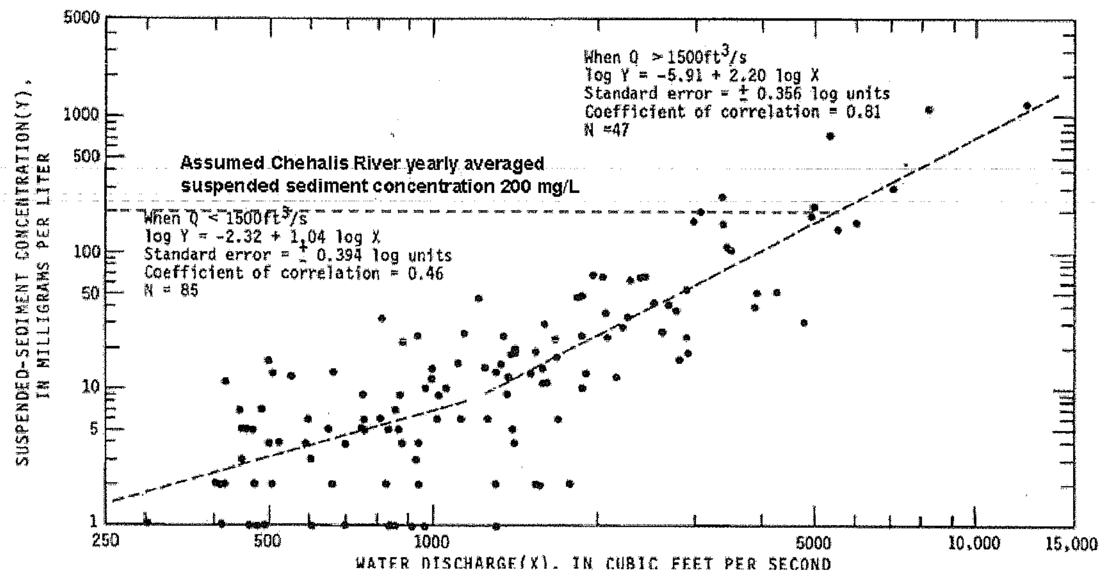

Figure 17. Statistical empirical relationship between flow discharge and suspended sediment concentration at the Calawah River (L. Nelson 1982)

Figure 17 shows actual long-term measured suspended sediment concentration data and corresponding flow discharge at Calawah River near Forks. The figure indicates certain statistical correlation between these two parameters (flow discharge and suspended sediment concentrations). One approach would be to extract suspended sediment concentration in the Calawah River at its annual average flow (1,040 cfs) and apply it for Chehalis River annual average flow (approximately 6,000 cfs). However, this approach may under-estimate evaluation of suspended sediment concentration in Chehalis River because of possible differences in fluvial morphologies between these rivers. A realistically conservative value was selected to compute sedimentation in the emergent part of the launch channel that encompasses most of the measured suspended sediment concentrations in Calawah River (excluding the four largest flow events). The recommended Chehalis River suspended sediment concentration is suggested to be approximately 200 mg/L.

Computation of sedimentation rate in the emergent part of the channel was conducted at first on a single tidal cycle basis, assuming that all suspended sediment in a water column would deposit at the bottom of the channel during each ebb and flood tidal cycle[9]. The computed volume of sediment per single cycle was then proportionally adjusted for a typical one-month period. Using the value of suspended sediment 200 mg/L as a constant throughout the year and considering deposition of all suspended sediment during each tidal cycle, a rate of sedimentation in the emergent channel was estimated at approximately 0.12 ft per month. The result of sedimentation rate estimate in the emergent channel is consistent with the small changes observed in the transects obtained from the prototype analysis for the Created Slough (see Section 2).

5. Launch Channel Sedimentation Summary and Recommendations

The recommended value of the sedimentation rate in the submerged part of the launch channel is estimated to be 0.72 ft per month during the primary period, and 0.24 ft per month during the secondary period. The recommended value of the sedimentation rate in the emergent part of the launch channel is estimated at 0.12 ft per month. A time series of total sediment accumulations in the submerged and emergent parts of the channel are plotted in Figure 18.

[9] This assumption is likely conservative. Six hours of flood or ebb may not be sufficient to settle all suspended sediment from the water column. Therefore, the rate of sedimentation in the emergent channel based on this assumption may be over-predicted.

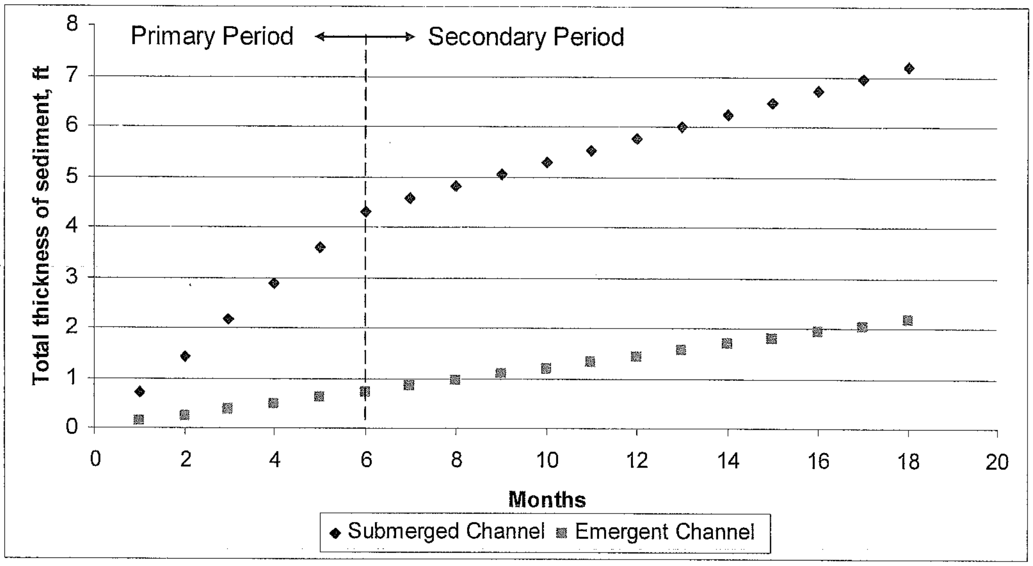

Figure 18. Time series of sediment accumulations in the submerged and emergent parts of the launch channel

The figure shows that the thickness of sedimentation in the submerged part of the channel would increase rapidly during the first six months (primary sedimentation) and more gradually during the rest of the year. Sedimentation at the emergent part of the channel would increase gradually at a slightly lower rate. The figure also shows a gradient in sediment deposits in the submerged and emergent parts of the channel. Because of excessive thickness and higher elevations, some sediment from the submerged part of the channel would propagate (by gravity) to the emergent part of the channel until stable equilibrium or natural angle of repose forms.

Based on the data from the prototype channels, it is assumed that the angle of repose for the deposited sediment (silt) is flat, and varies from approximately 7H:1V to 50H:1V. For purpose of calculations a 30H:1V slope is assumed. Using these assumptions and estimates of the sedimentation rate for the submerged and emergent parts of the channel, a longitudinal profile of predicted sedimentation along the launch channel for a one-year period with no maintenance dredging was computed and is plotted in Figure 19.

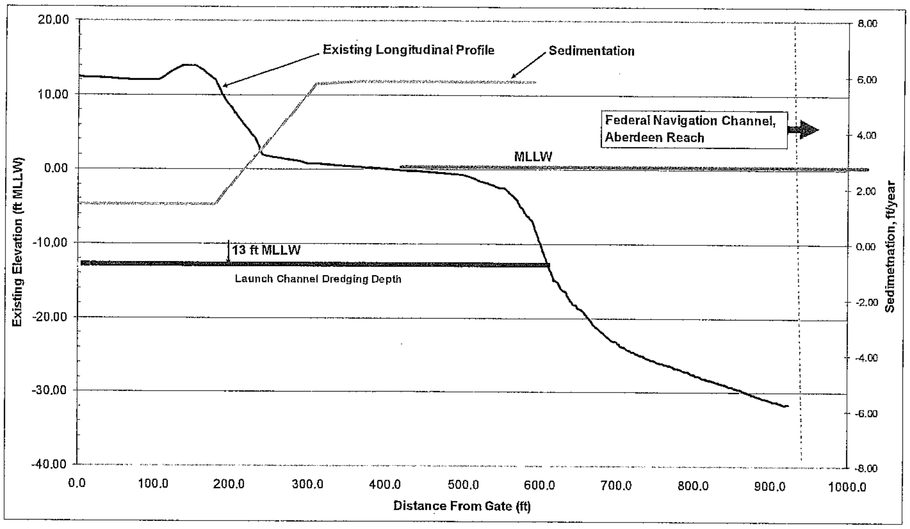

Figure 19. Longitudinal profile of predicted sedimentation along the launch channel

The figure shows that channel sedimentation would occur at different rates along the channel centerline. The total thickness of sedimentation in the submerged channel would be 5.8 ft per year. Total sedimentation in the emergent channel is estimated to be approximately 1.4 ft per year. A variable thickness of sediment deposition would occur along the transition zone between the submerged and emergent parts of the channel.

5.1. Maintenance Dredging Requirements

A typical design of a navigation channel defines the design dredging depth consisting of two major components: (1) navigable depth, and (2) clearance on advanced maintenance dredging (also referenced as over-dredge allowance, or simply channel over-dredging). Navigable depth is a minimal required depth to provide safe navigation for the design vessel that includes clearances on vessel motion, trim, safety factors, tide, and so forth. Channel depth shall not be less than navigable depth at any time specified for a channel use.

Advanced maintenance dredging is a clearance for channel sedimentation. It is included so that the required navigable depth is maintained if the channel is subjected to sedimentation. Theoretically, advanced maintenance dredging should be equal to a rate of sedimentation that may occur between sequential maintenance dredging events.

The advanced maintenance dredging clearance was not specified originally for the launch channel design. The analysis considered a channel bottom elevation of -13.0 ft mean lower low water (MLLW), per the preliminary design drawings. Upon preliminary analysis and in coordination with the Project Team, it was established that the minimum navigable depth in the launch channel is 9.0 ft at MLLW (elevation -9.0 ft MLLW). Because the design channel bottom elevation is below the required navigable elevation, the channel can accommodate 4.0 ft of sedimentation without compromising the navigable depth of the channel. Therefore this 4.0 ft clearance can be effectively considered as advanced maintenance dredging.

Based on the estimated rates of sedimentation, it appears that the emergent section of the channel would provide the required navigation depth for more than 12 months. Maintenance dredging in this part of the channel may be required approximately once every two or three years, based on a volume of approximately 2,200 CY per year of accumulation.

It appears that the submerged section of the channel would provide the required navigation depth for approximately five to six months, and then maintenance dredging would be required, with a corresponding volume of approximately 16,000 CY per every six months.

One approach to deal with the sediment would be to conduct maintenance dredging work to maintain the navigable depth at the submerged channel at 6 (six) month intervals, or less frequently if conditions warrant. In this case, the channel would be constantly subjected to a primary sedimentation period that would require frequent maintenance dredging.

Alternatively, it may be advantageous to increase the initial advance maintenance dredging cut in the submerged part of the channel. For example, the frequency of dredging would be reduced to one time every 9 (nine) months if this advanced maintenance dredging cut depth is increased by 1 (one) foot.

We recommend using the approach for increased advance maintenance dredging only after completion of a channel monitoring program for the first six months after construction (first-time dredging). The monitoring program would include frequent hydrographic surveys and would provide information to ground truth predictions of sedimentation and help optimize the advance maintenance dredging cut elevation, based on actual experience at the project site. It is also possible that due to the softness of silting material, sediment deposition would be distributed more evenly over the bottom slope, rather than forming a transition slope between the submerged and emergent parts of the channel. This would result in leveling off of the highest sedimentation (controlling section) and less frequent overall maintenance dredging. This and other specifics, however, should be determined upon initiation of the monitoring program.

Sedimentation in the launch channel would not occur uniformly in time, but rather would vary on scales from hours to months dependent on actual conditions. Estimated sedimentation rates and patterns described herein are considered typical average rates and suitable for planning purposes. Monthly rates of sedimentation herein are derived from the average annual rates, and are not intended as precise estimates. Storm events can alter short-term (monthly) and longer-term (annual) rates of sedimentation at the project site.

6. References

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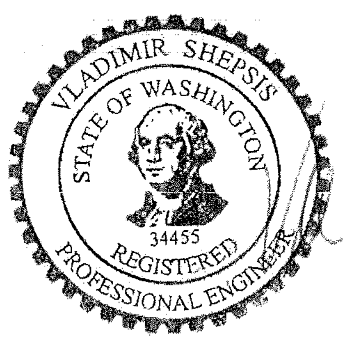
TECHNICAL REPORT VOLUME 2

SR520 COASTAL ENGINEERING REPORT
GRAYS HARBOR SITE COASTAL HYDRAULIC AND GEOMORPHOLOGIC CONDITIONS

This document was prepared by a Professional Engineer.



Vladimir Shepsis, Ph.D., P.E.
Principal, Coast and Harbor Engineering

COAST & HARBOR
ENGINEERING

110 Main Street, Suite 103
Edmonds, WA 98020
Ph 425 778.6733
Fax 425 977.7416

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SR520 Coastal Engineering Report – Grays Harbor Site Coastal Hydraulic and Geomorphologic Conditions

Executive Summary

This report is Volume 2 of a three-volume Technical Report prepared by Coast & Harbor Engineering, Inc. (CHE) under Task Order 1 for the KIEWIT-HNTB SR 520 Pontoon Construction D-B Project and presents the results of work completed under Task 2 of the Scope of Work, Grays Harbor Site Coastal Hydraulic and Geomorphologic Conditions. Volume 1 of the Technical Report presents the results of analysis and numerical modeling conducted for Task 1 of the Scope of Work, Channel Sedimentation Study. Volume 3 of the Technical Report presents the results and analysis for Task 3, Launch Channel Side Slope Rock Dimensions, and for Task 6, Dolphins Scour Analysis and Protection Design.

The objective of this report is to document existing hydraulic (tides, currents, and waves) and geomorphology (sediment transport, erosion/accumulation) conditions in the vicinity of the project area, and possible changes of existing conditions due to construction and maintenance of the navigation channel (project).

The existing tidal and river-driven currents and possible change to these currents that may occur due to construction of the launch channel was conducted using the 3-Dimensional flow circulation numerical model SELFE (Baptista et al. 2005). Modeling was conducted for existing conditions (without launch channel) and project conditions (with launch channel) separately for the same input parameters and modeling (design) scenarios. Results of the modeling were compared, and the differences in current velocities between existing and project conditions were quantified to evaluate possible impacts from the launch channel.

Prior to the modeling of existing and post-project conditions, the SELFE numerical model was validated with field measurement data. The validation showed that numerical modeling results are of sufficient accuracy to evaluate existing conditions and provide a relative comparison between existing and project conditions.

Two methods were applied to detect and evaluate potential changes of current velocities due to construction of the proposed launch channel. The first method (spatial) calculated the difference of current velocities between existing conditions and project conditions at the same time in the simulation at each computation point in the model. The second method of analysis (time domain) consisted of extracting current velocities from the reference stations at the same locations that were used for documenting existing conditions. Both methods of analysis determined that either no change or a small change of velocities may occur upon construction of the launch channel in the project vicinity. These methods also determined that no detectable

changes of velocities will occur outside of the project area after launch channel construction for either day-to-day or extreme flow conditions in the Chehalis River.

Wave conditions at the project area were documented based on compilation and processing of historical wind and tide data and wave numerical modeling of existing conditions (without launch channel) using the spectral 2-Dimensional (2-D) numerical wave model SWAN. Possible changes to wave conditions due to construction of the launch channel were investigated by numerical modeling (the same SWAN model) of project conditions (with the launch channel), and conducting a comparison with the modeling results for existing conditions.

Because upper Grays Harbor wave measurements data was not available, SWAN model validation at the project site was not conducted. Due to previous experience with this model and preceding wave model validation at other similar projects in Puget Sound, it is suggested that the model is a reliable engineering tool for the purpose of this study.

Upon modeling and comparison, it was determined that existing wave conditions at the project site are very mild. Significant wave height at the site is relatively small, generally less than 3 ft even during the most extreme conditions. Based upon numerical modeling, no change or very small changes to existing wave heights (less than 0.1-0.2 ft) would occur upon construction of the launch channel. If these small changes to wave height occur, changes would be expected in the vicinity of the project area, just east of the launch channel due to the predominant wind from the west.

Existing geomorphologic conditions at the project site were documented based on compilation, processing, review, and analysis of the existing data, previous study results, and practical experience with coastal projects in the Grays Harbor Estuary area. Possible changes to geomorphic conditions due to the project were identified as a function of possible changes in hydraulic conditions and results of the Channel Sedimentation Study at the proposed launch channel, presented in Volume 1 of the report. Two possible impacts on existing geomorphologic conditions from construction of the launch channel have been considered during analysis herein: direct and indirect. Considering that a limited and small area of possible alteration of existing hydrodynamic conditions is expected, the type and relatively small volume of sediment involved in sedimentation of the proposed channel, no significant direct or indirect impacts to the existing geomorphic conditions will occur. For a project lifetime on the order of decades (10 to 50 years), the project life is short compared to geomorphic and geologic timescales and therefore it is concluded that no significant direct or indirect impacts to the long term geomorphic conditions will occur due to construction and maintenance of the launch channel.

COAST & HARBOR ENGINEERING

Technical Report Volume 2
SR520 Coastal Engineering Report – Grays Harbor Site Coastal Hydraulic and Geomorphologic Conditions

1. Introduction

This report is Volume 2 of a three-volume Technical Report prepared by Coast & Harbor Engineering, Inc. (CHE) under Task Order 1 for the KIEWIT-HNTB SR 520 Pontoon Construction D-B Project. The launch channel would be dredged and excavated to facilitate navigation operations for towing pontoons (SR 520 Bridge Replacement and HOV Project) from the casting basin and into the Federal Navigation Channel. The pontoons would then be towed further to a pre-determined moorage location.

The objective of this report is to document existing hydraulic (tides, currents, and waves) and geomorphologic (sediment transport, erosion/accumulation) conditions in the vicinity of the project area, and possible changes in existing conditions due to construction and maintenance of the navigation channel (project).

The hydraulics and geomorphology of the project area are described herein to develop a basis for the evaluation of existing conditions, and possible changes that may occur with the construction and maintenance of the pontoon launch channel (project conditions). Project conditions for the launch channel were taken directly from preliminary electronic design drawings (CAD files) provided by HNTB Corporation in July 2010. The launch channel that was incorporated into the numerical models had the following characteristics:

- A total length of approximately 610 ft from casting basin gate to end of channel.
- A channel bottom width ranging from 138 ft to 340 ft per CAD files.
- A bottom elevation of -13 ft MLLW.
- Channel side slopes of 3:1 (H:V) in both the emergent launch channel near the casting basin and 3:1 (H:V) in the submerged part of the launch channel.

The following sections of this report present the general approach and methodology of the analysis and include major input parameters that were used for the analysis. Also included is documentation for existing tides, currents, waves, and geomorphologic conditions and descriptions of possible changes to these conditions due to construction of the launch channel.

2. Currents - Existing and Project Conditions

2.1. Approach

The existing tidal and river-driven currents and possible changes to these currents that may occur due to construction of the launch channel were analyzed using the 3-Dimensional flow circulation numerical model SELFE (Baptista, 2005). Modeling was conducted for existing (without launch channel) and project (with launch channel) conditions separately for the same input parameters and modeling (design) scenarios. Results of the modeling were compared and differences in current velocities between existing and project conditions were quantified to evaluate possible impacts from the launch channel.

Numerical modeling was conducted on a large numerical modeling domain extending offshore into the Pacific Ocean at approximately 150 miles, and included the entire Grays Harbor and the lower Chehalis River areas. The extent of the modeling domain and resolution of the modeling mesh are shown in Figure 1[1].

Prior to the modeling of existing and project conditions, the SELFE numerical model was validated with field measurements data. A more detailed discussion on model validation is presented below.

2.2. Input Parameters

Major input parameters for SELFE numerical modeling included tide elevations, Chehalis River flow discharge, and bathymetry, including detailed survey data at the project area.

2.2.1. Tide Elevations

Tide elevations were input as variable parameters along the model offshore boundaries (see Figure 1) to properly represent propagation of the tidal wave (ebbs and floods). Tide elevations were input into the model as constituents of tidal harmonics obtained from NOAA predictions for each modeling period (see References).

2.2.2. River Discharge

Chehalis River flow discharge data were derived from the nearest USGS measuring stations (see References) and from a previous FEMA flood study (FEMA, 1984). Long-term USGS flow measurements along the lower Chehalis River are available at Porter[2]. Typical monthly Chehalis River flows at Montesano (50 percent exceedance[3]) and daily mean flows at the Porter station are provided in Table 1.

[1] Note the coarse modeling grid (scale in miles) at the open ocean (2a) and detailed fine numerical modeling grid (scale in feet) in the vicinity of the project area (2b).
[2] Daily mean flows from the USGS Station at Porter are available for the period from 1953 to 2009.
[3] Statistical flow discharge data at Montesano area adapted from previous studies (Envirovision 2000).

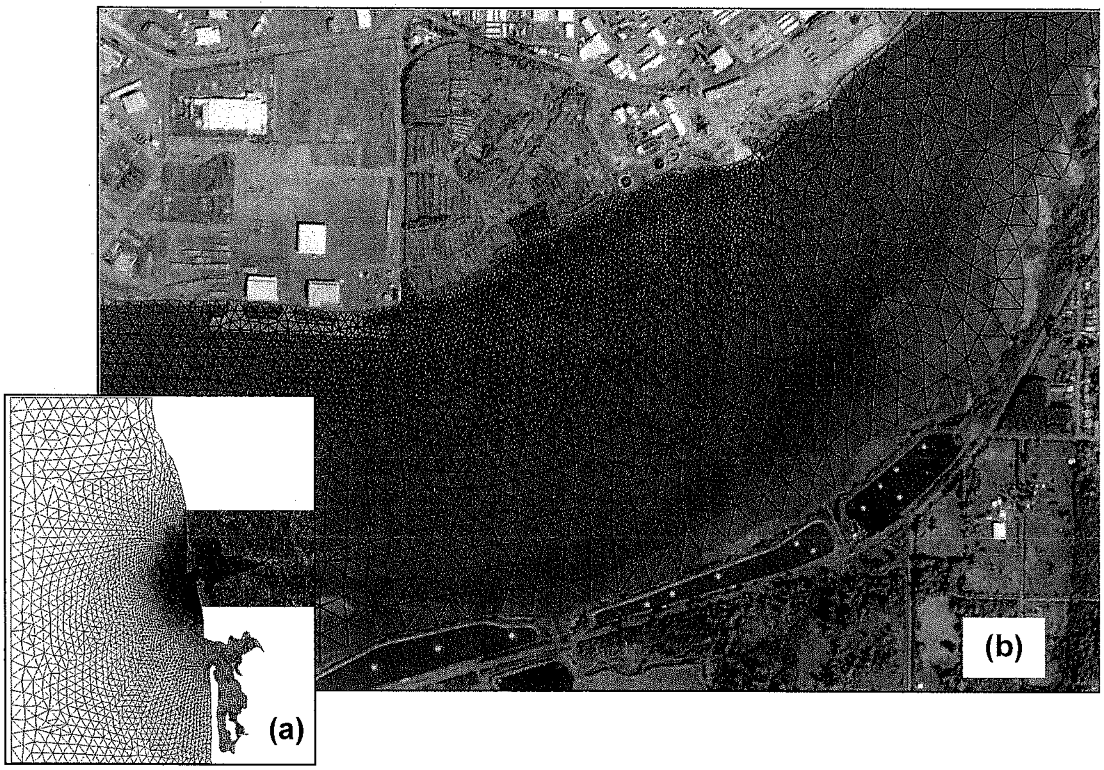

Figure 1. Image of (a) full tidal model and (b) zoomed in at project site numerical model mesh

Table 1. Chehalis River flows (based on USGS data)

Month	50 Percent Exceedance at Montesano, WA[3] (cfs)	Daily Mean Flow at Porter, WA[2] (cfs)
October	2,078	1,231
November	8,296	5,201
December	13,144	8,866
January	13,445	9,520
February	12,987	8,198
March	10,260	6,512
April	6,853	4,374
May	3,761	2,136
June	2,124	1,198
July	1,333	608
August	915	414
September	985	533

The 50 percent exceedance data at Montesano were applied for the modeling because the data incorporated inflow from two additional tributaries downstream of the Porter station. From the table, typical flows are an order of magnitude greater in the winter months than in the summer months. Two typical river discharge cases were selected for numerical simulation: day-to-day winter flow conditions and day-to-day summer flow conditions. The winter flow was input into the model at 13,500 cfs. The summer flow was input at 1,060 cfs (average between three months (July, August, and September). Both summer and winter flows were steady during the periods of simulation.

The effective FEMA Flood Insurance Study was used to determine input parameters for Chehalis River flow modeling of an extreme event. The FEMA study provided extreme discharge rates for the Chehalis River near the project site, as summarized in Table 2.

Table 2. Return period Chehalis River discharges (FEMA 1984)

Flooding Source: Chehalis River	Peak Discharge (cubic feet/sec)			
	10-Year	50-Year	100-Year	500-Year
Chehalis River Mouth	55,000	70,500	77,000	102,200

The 25-year return period flow at the Chehalis River was selected as an extreme event for flow modeling. The flow discharge for this event was obtained by interpolation from the FEMA discharge data in Table 2, yielding a peak discharge of 63,800 cubic feet/second (cfs).

2.2.3. Bathymetry

Various sources of bathymetric data were compiled to construct a bathymetric model of Grays Harbor, as summarized in Table 3. Where bathymetric datasets overlapped, these datasets were compared, and the most recent or complete data were incorporated into the numerical modeling. Because much of the data available has been compiled by others from miscellaneous sources, actual survey data were given priority over gridded digital elevation model information.

Table 3. Bathymetry data sources used

Source	Coverage	Date(s)
HNTB	Project site mudflats, uplands, Fed. Channel	2008-2010
USACE[1]	Federal Navigation Channel (intermittent)	2000 to 2009
Golder[2]	Project site nav. channel, misc in Grays Harbor	May to June 2008
NOAA[3]	Fed. Channel from Cow Point to Aberdeen	March 2004 to July 2005
NOAA	Digital elevation model of Grays Harbor	Various Dates
USACE	ADCIRC model domain from 2008 study	Various Dates

Notes:
[1]U.S. Army Corps of Engineers
[2]Golder Associates
[3]National Oceanic and Atmospheric Administration

The modeling grid in the vicinity of the project area included detailed bathymetric survey data provided by HNTB, as shown in Figure 2. In the figure blue colors indicate greater water depth.

Figure 2. SELFE modeling grid elevations in vicinity of project area

2.3. Model Validation

SELFE model validation was conducted to determine reliability of the modeling results in evaluating existing conditions, and to detect possible changes to flow circulation that may occur with construction of the launch channel. The flow model was validated using measured current velocities at the project site (provided by Kiewit) during the period from August 26 to September 30, 2010. Figure 3 shows the approximate location of the current meter (red dot) with respect to the proposed launch channel footprint (white lines). The assumed vertical position of the measuring device (Marsh-McBirney FLO-TOTE sensor) relative to the existing bottom elevation is illustrated in Figure 4.

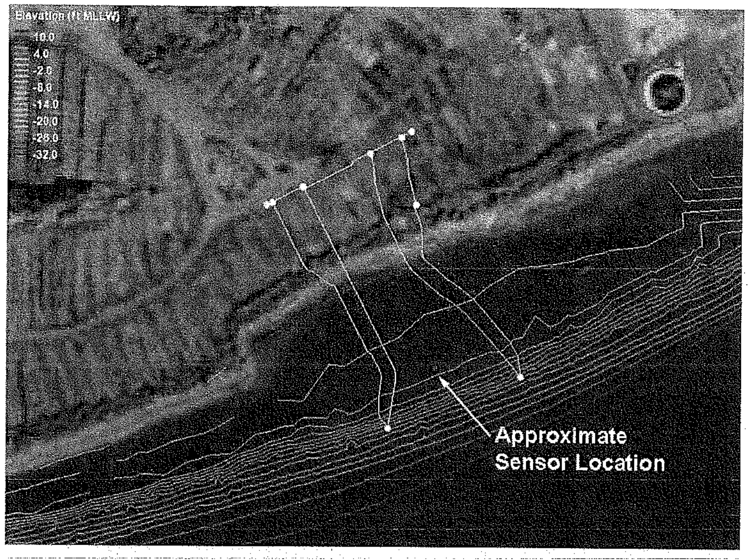

Figure 3. Approximate location of current meter (red dot) with respect to proposed launch channel footprint (white lines)

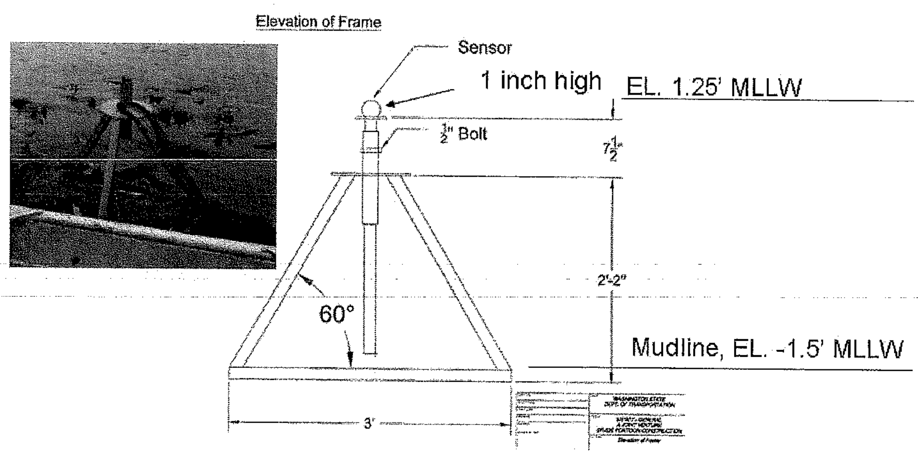

Figure 4. Vertical position of sensor with respect to existing mudline (drawing and photograph provided by Kiewit)

From the sketch provided by Kiewit and based upon its location, CHE estimated the sensor is located approximately 1.25 ft above MLLW, and thus, was periodically exposed (dry) during the period of measurement. Therefore, reliable measurements of current velocities are not available at low tide conditions, when the tide elevation drops below approximately 1.5 ft MLLW.

The numerical model was run for the beginning of the current measurement period, from August 28 to September 2, 2010. During that period, measured Chehalis River discharges at the Porter station were steady and typical of summer discharges. Current velocities were extracted from the numerical model at the same location as measured by the sensor in the field. The extracted modeling results were then compared to the field data. Figure 5 compares the measured current velocities to a time series of current velocities simulated by the model.

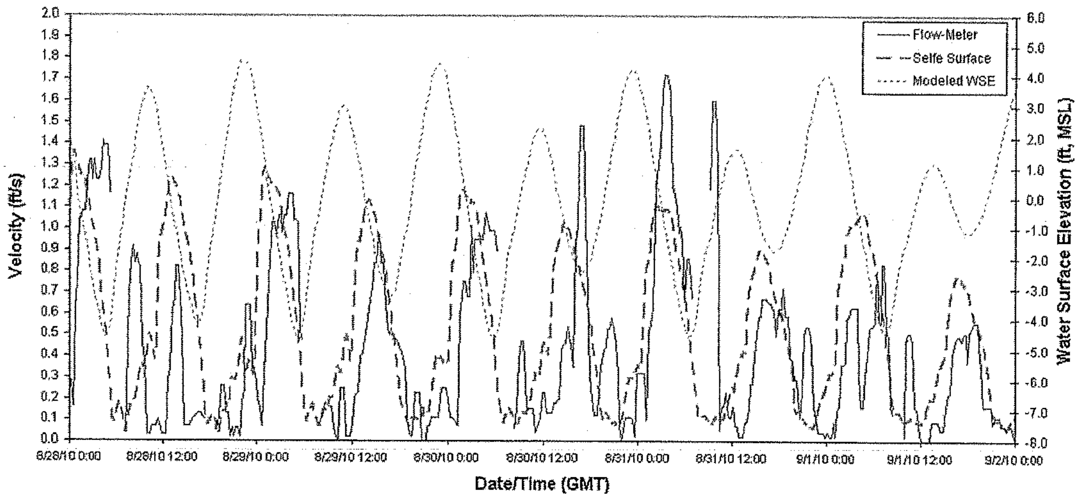

Figure 5. Modeled SELFE surface currents and currents measured by Kiewit at project site for August 28 through September 2, 2010

A comparison of the modeling and measured current velocities was conducted for middle and high tide conditions only, since the sensor was periodically dry at low tides. The figure above shows that current velocities simulated by SELFE are in close proximity to the measured data for most of the period of validation. The comparison analysis suggests that current velocities produced by the model are of sufficient accuracy for evaluation of existing conditions, and are suitable for providing a relative comparison between existing conditions and project conditions.

2.4. Existing Conditions and Analysis of Potential Changes

2.4.1. Existing Conditions

Once validated, the model was used to determine and document existing conditions river-tidal flows at the project site. Modeling of existing conditions was conducted for day-to-day (summer and winter) and extreme flow conditions. Examples of modeling results for day-to-day winter and summer conditions during maximum flood and maximum ebb are shown in Figures 6 and 7, respectively. An example of the modeling results for extreme conditions, maximum flood and maximum ebb, are shown in Figure 8.

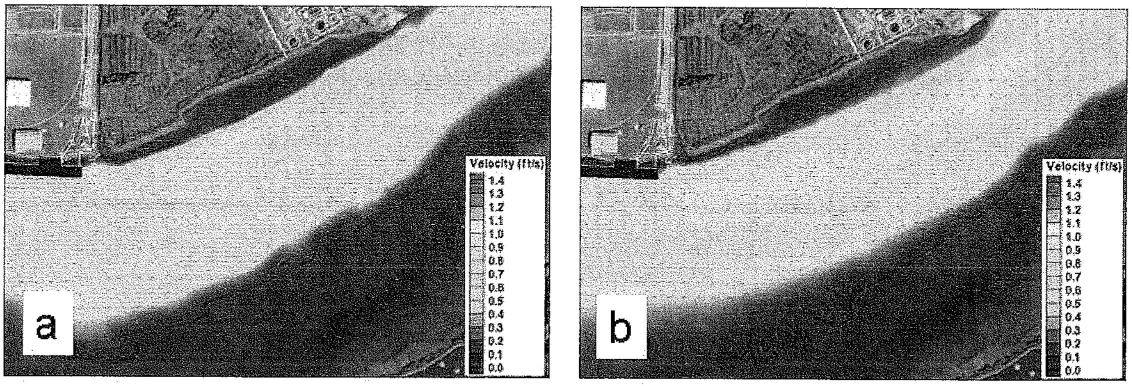

Figure 6. Example of modeling results for day-to-day winter conditions, a) maximum flood and b) maximum ebb

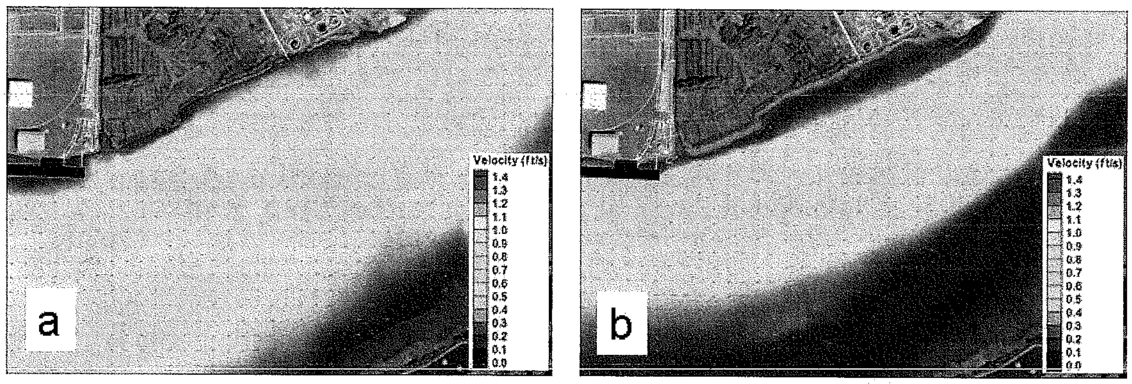

Figure 7. Example of modeling results for day-to-day summer conditions, a) maximum flood and b) maximum ebb

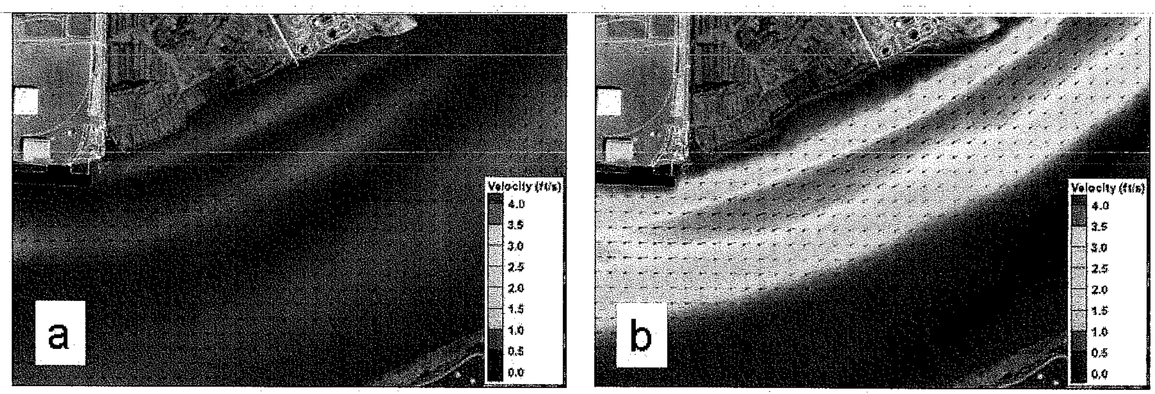

Figure 8. Example modeling results for 25-year conditions, a) maximum flood and b) maximum ebb

The figures show, in color formats, snap-shots of depth-averaged velocities in the vicinity of the project area. The depth averaged velocity is the average velocity of the water column. Red color corresponds to a higher velocity and blue color to lower velocity. To document flow velocities for existing conditions, model gauges were installed along the modeling domain, and maximum flow velocities were extracted for each modeling scenario. The location of the model gauges are shown in Figure 9. Representative extracted modeling results are presented in Table 4. Velocities of zero (0.00 ft/s) indicate that the location is dry when maximum flood or ebb occurs in the Federal Navigation Channel.

Figure 9. Location of model gauges used to document model current velocities for existing conditions

Table 4. Depth-averaged current velocities for simulated existing conditions

		P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12
		Velocity (ft/s)											
Summer	Max Flood	1.25	0.73	0.80	0.02	0.00	0.03	0.78	0.06	0.00	0.78	0.78	0.76
	Max Ebb	0.81	0.70	0.74	0.28	0.54	0.13	0.77	0.48	0.26	0.79	0.79	0.73
Winter	Max Flood	0.62	0.80	0.86	0.00	0.00	0.00	0.82	0.00	0.00	0.78	0.90	0.82
	Max Ebb	1.45	0.80	0.87	0.00	0.00	0.00	0.87	0.00	0.00	0.89	0.89	0.86

2.4.2. Project Conditions Impact Analysis

This section describes the potential changes to the existing patterns of tidal- and current-driven circulation due to the construction of the launch channel. The launch channel was incorporated into the numerical modeling grid, as discussed above in Section 2, and the modeling was repeated for the same modeling scenarios as for existing conditions. An example of project modeling results in comparison to existing conditions for the summer simulation period is shown in Figure 10.

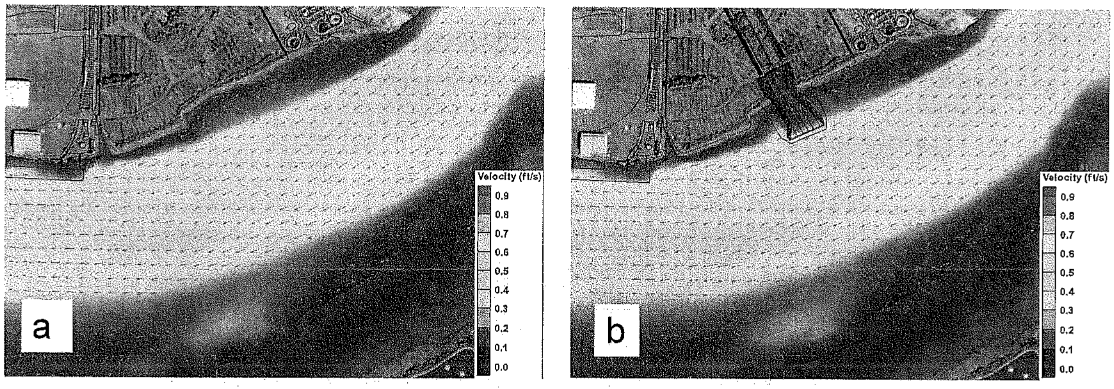

Figure 10. Launch channel simulated depth-averaged velocities for maximum ebb currents for summer period for (a) existing conditions (b) and project conditions

Two methods were applied to detect and evaluate potential changes of current velocities due to construction of the proposed launch channel. The first method (spatial) calculated the difference of current velocities between existing conditions and project conditions at the same time in the simulation at each computation point in the model. Representative spatial difference plots of velocities are shown in Figures 11 through 13.

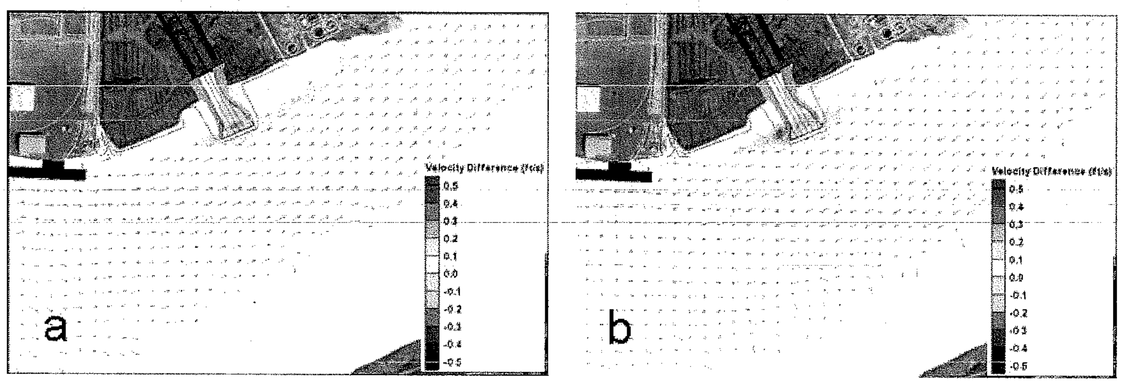

Figure 11. Depth-averaged velocity differences plot (project minus existing) of (a) peak flood currents and (b) peak ebb currents during winter period

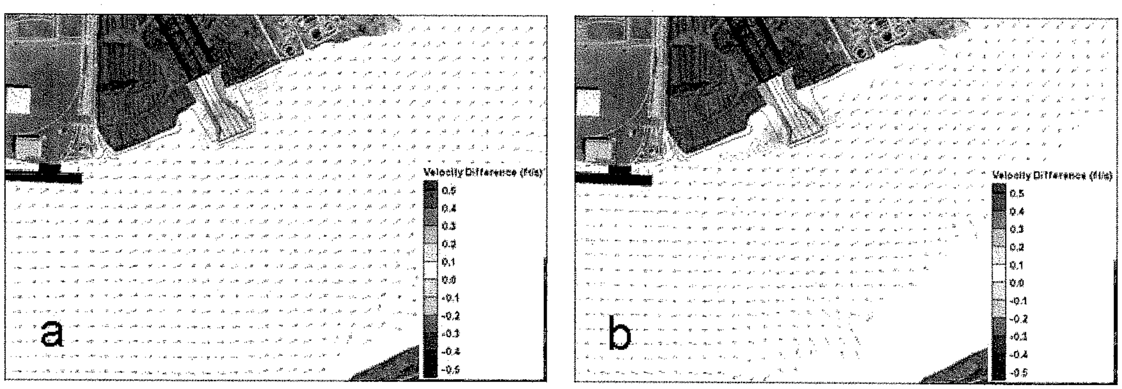

Figure 12. Depth-averaged velocity difference plot (project minus existing) of (a) peak flood currents and (b) peak ebb currents during summer period

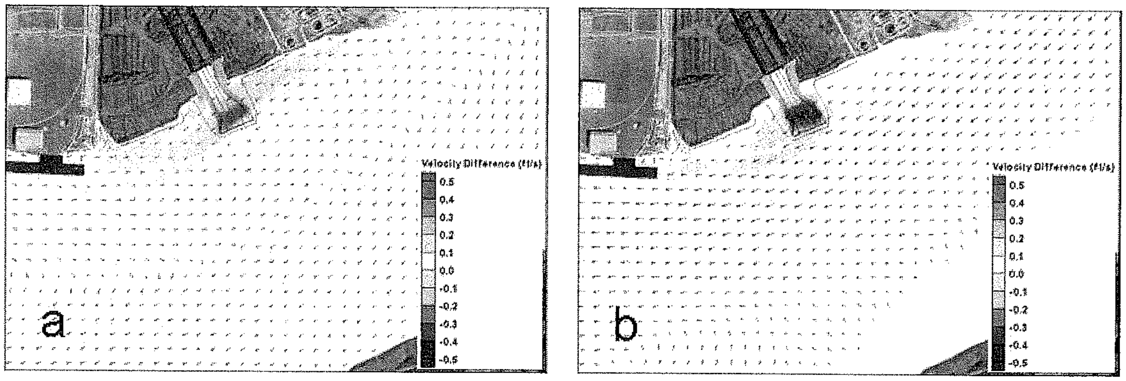

Figure 13. Depth-averaged velocity difference plot (project minus existing) of (a) peak flood currents and (b) peak ebb currents during extreme event

In the difference plots, red spectrum colors indicate an increase of velocities, blue color indicates a reduction of velocities, and white color indicates no change of velocities between existing and project conditions. The figure shows a relatively small change of velocities in the vicinity of the project area. As expected, the velocities within the launch channel will be lower than for existing conditions due to the effects of increased water depth in the launch channel. Peak tidal velocities will increase slightly (about 0.1 ft/s) on either side of the project launch channel due to the abrupt change in depth at the launch channel side slopes.

At distances exceeding 1,300 ft from the launch channel, changes in velocity are undetectable in the model and are expected to be negligible. No changes in current velocities were detected by the model for all examined scenarios (summer, winter, extreme) outside of the project vicinity, in the Federal Navigation Channel, and at the Port of Grays Harbor T4 Dock.

The second method of analysis (time domain) consisted of extracting current velocities from the reference stations (the same model gauge stations that were used for documenting existing conditions shown in Figure 9) and comparing existing conditions and project conditions. An example of comparing a time series of current velocities at two stations for existing and project conditions for the summer period modeling scenario is shown in Figure 14. Water surface elevation (tide level) is also shown in the figure. Figure 14 (a) shows, as expected, a slight decrease of depth-averaged flow velocities in the launch channel due to increased project depth. Figure 14 (b) shows no change in current velocities seaward of the launch channel. Similar results (small change of velocities in the project vicinity) and no detectable change of velocities just outside of the project area were observed for all modeling scenarios (winter, summer, and extreme event).

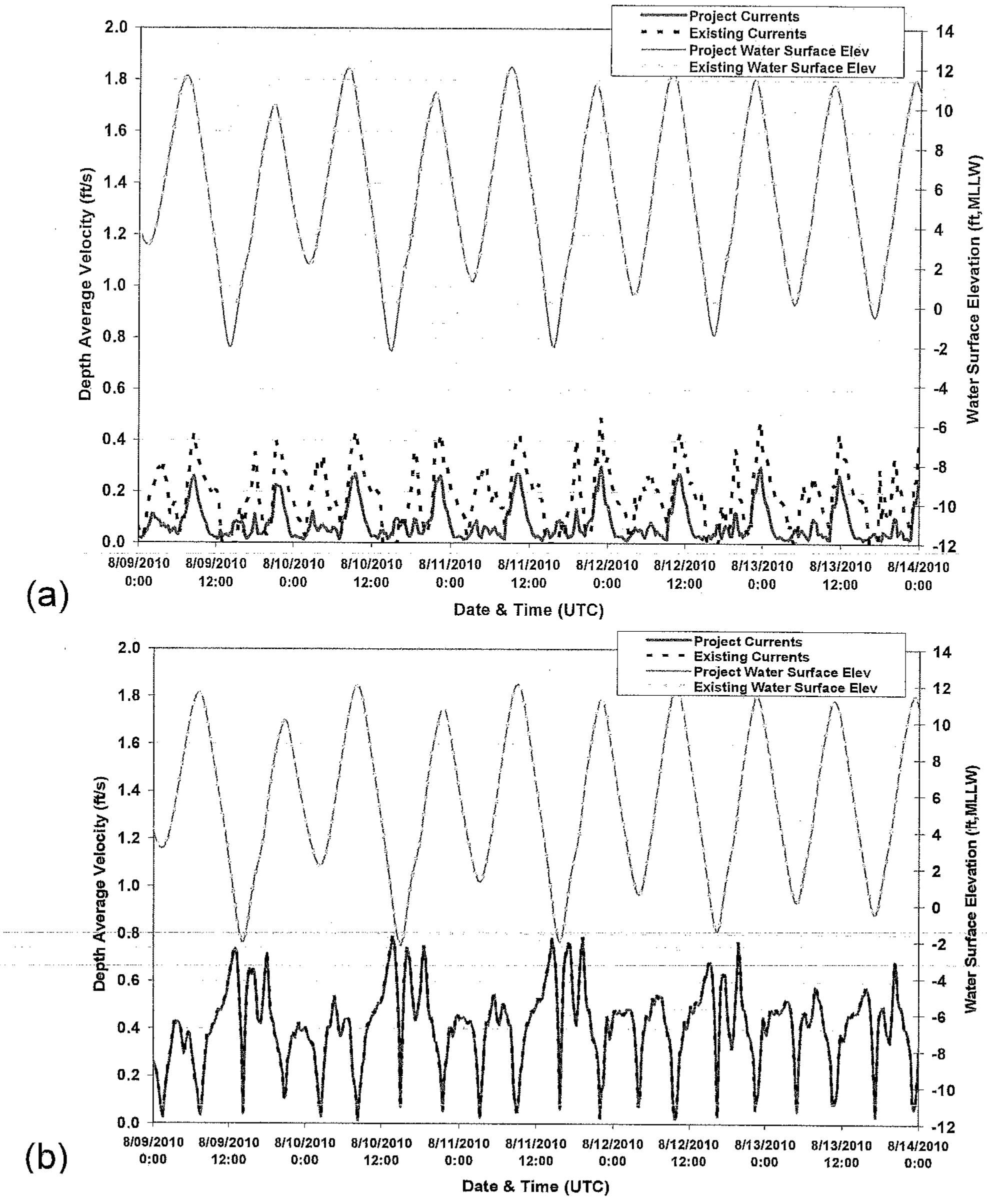

Figure 14. Depth-averaged velocity time series at (a) Point 6 located within the launch channel and (b) Point 7 outside the launch channel for the summer period

3. Waves - Existing and Project Conditions

3.1. Approach

Modeling of existing wave conditions and possible changes to these conditions that may occur due to construction of the launch channel was conducted using the spectral 2-D numerical model SWAN (Simulating WAves Nearshore). Modeling was conducted for existing (without launch channel) and project (with launch channel) conditions separately for the same input parameters and modeling scenarios. Results of the modeling were compared, and the differences in wave heights between existing and project conditions were quantified to evaluate possible impacts from the launch channel.

Waves were generated using a large-scale wave model grid covering the entirety of Grays Harbor. A finer-scale nested SWAN model was applied to simulate wave growth and transformation in the vicinity of the project site. The large modeling domain and nested domains are illustrated in Figure 15; model water depths are shown (relative to mean sea level).

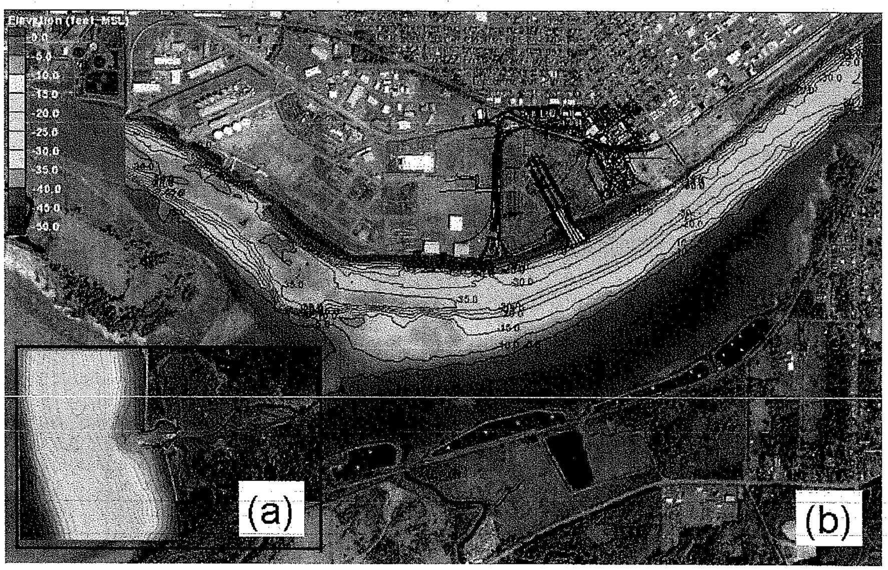

Figure 15. SWAN wave model (a) large-scale domain (100x100m cell size) and (b) nested (10x10m cell size) domain used in the wave model

No wave measurements at the upper Grays Harbor Estuary area were identified during the study. Therefore, no SWAN model validation was conducted at the project site. However, previous experience with wave modeling and prior wave model validation at other similar projects in Puget Sound suggest the model is a reliable engineering tool for the purpose of this study.

3.2. Input Parameters

Major input parameters for SWAN numerical modeling included tide elevations, wind speed and direction, and bathymetry[4].

[4] Bathymetry data was incorporated into the wave model in a similar manner as discussed in Section 2.2 for SELFE modeling; therefore, it is not discussed further.

3.2.1. Tide Elevations

Tidal datums are based on NOAA published records for the 1983-2001 tidal epoch at the Westport, WA and Aberdeen, WA Stations, and are summarized in Table 5. Tidal datums at Aberdeen are considered representative for the project site. Tides are mixed semi-diurnal, with a diurnal tide range of 10.11 ft and mean tide range of 7.94 ft at Aberdeen. Figure 16 shows the relationship within the tidal and geodetic datums which are applicable to the project site.

Table 5. Tidal datums in Grays Harbor

TIDAL DATUMS Tidal Epoch: 1983-2001	Westport Sta. 9441102	Aberdeen Sta. 9441187
	Elevation (ft MLLW)	Elevation (ft MLLW)
HIGHEST OBSERVED WATER LEVEL (11/06/2006,12/03/1982)	12.06	13.86
MEAN HIGHER HIGH WATER (MHHW)	9.05	10.11
MEAN HIGH WATER (MHW)	8.32	9.41
MEAN TIDE LEVEL (MTL)	4.85	5.44
MEAN SEA LEVEL (MSL)	4.83	5.60
MEAN LOW WATER (MLW)	1.37	1.47
MEAN LOWER LOW WATER (MLLW)	0.00	0.00
LOWEST OBSERVED WATER LEVEL (05/17/2007, 1982/07/22)	-3.39	-3.35

Aberdeen, WA NOAA Station 9441187

Datums Applicable to Pontoon Construction Project Site

	MLLW	NGVD29	NAVD88
Highest Observed Water Level	13.86	8.98	12.27
Mean Higher High Water	10.11	5.23	8.52
Mean High Water	9.41	4.53	7.82
Mean Tide Level	5.44	0.56	3.85
National Geodetic Vertical Datum	4.88	0.00	3.29
North American Vertical Datum	1.59	-3.29	0.00
Mean Low Water	1.47	-3.41	-0.12
Mean Lower Low Water	0.00	-4.88	-1.59
Lowest Observed Water Level	-3.35	-8.23	-4.94

Epoch 1983 - 2001

Figure 16. Tidal and geodetic datums at Aberdeen, WA

The MHHW tide elevation equal to 10.11 ft MLLW was selected as an input parameter for all wave modeling scenarios. The MLLW tide elevation was also modeled, but resulted in very small waves due to shallow water depths throughout Grays Harbor.

3.2.2. Wind Data

Wind data at Bowerman Field (airport) at Hoquiam, WA for the period from 1973-2009 were compiled and analyzed to develop input parameters for wave modeling[5]. An annual wind rose based on these data was computed and plotted, as shown in Figure 17.

[5] The anemometer at the wind measuring station has an unobstructed exposure close to the shoreline and provides wind data that are representative of wave-generating winds within Grays Harbor.

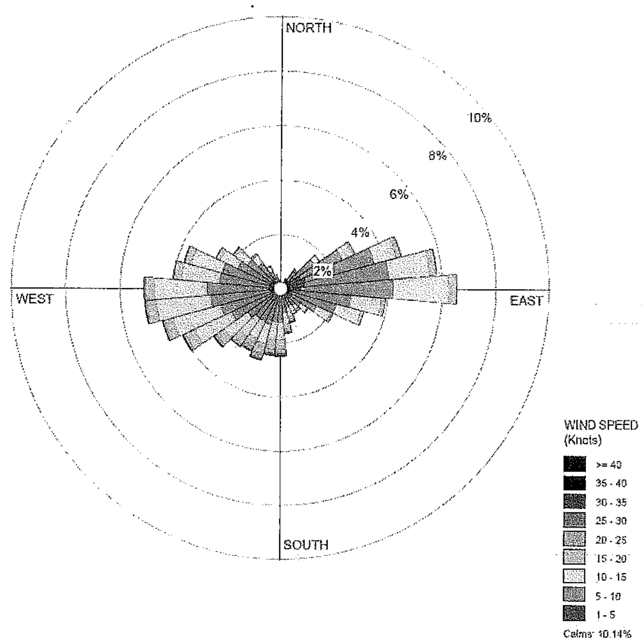

Figure 17. Annual wind rose plot for Bowerman Field, WA

The return period wind speeds were computed for extreme values for every 10 degrees of wind direction, and are shown in Figure 18[6]. Directions in both Figure 17 and Figure 18 are the directions from which the wind blows.

The wind input parameters for wave modeling were selected for storm events of 2-, 25-, and 50-year return periods. These winds approach the project site from the southwest to west directions. The wind input parameters are shown in Table 6.

[6] 37 years of a wind database are statistically reliable for providing predictions of extreme winds for a 100-year return period.

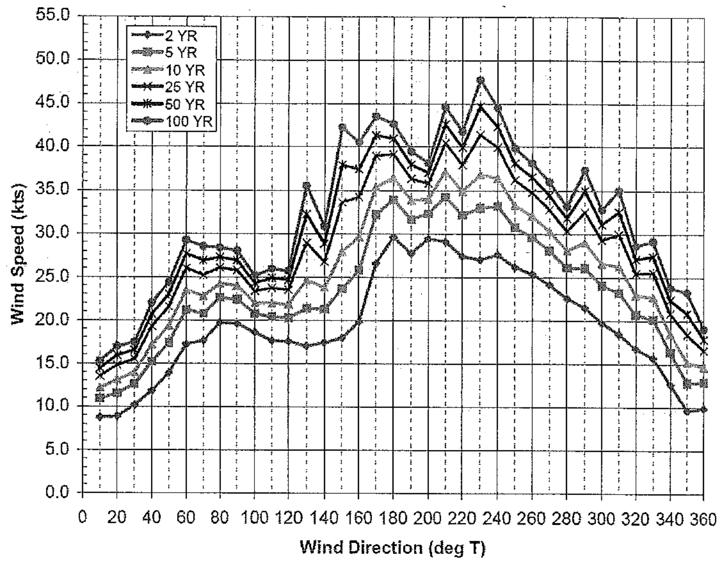

Figure 18. Return period 2-minute wind speed and direction for Bowerman Field

Table 6. Wave simulation cases for comparing existing and project conditions

Return Period	Wind Direction and Wind Speed (kts)		
(yr)	230	250	270
2	27.0	26.2	24.1
25	41.4	36.2	32.8
50	44.7	38.1	34.5

3.3. Existing Conditions and Analysis of Potential Changes

3.3.1. Existing Conditions

As discussed above, the modeling of existing conditions was conducted for 2-, 25-, and 50-year return period storm events from three different directions. Examples of the modeling results on the large and nested modeling domains for a 2-year return period storm event propagating from the southwest (230^0 True North) are shown in Figures 19 and 20, respectively.

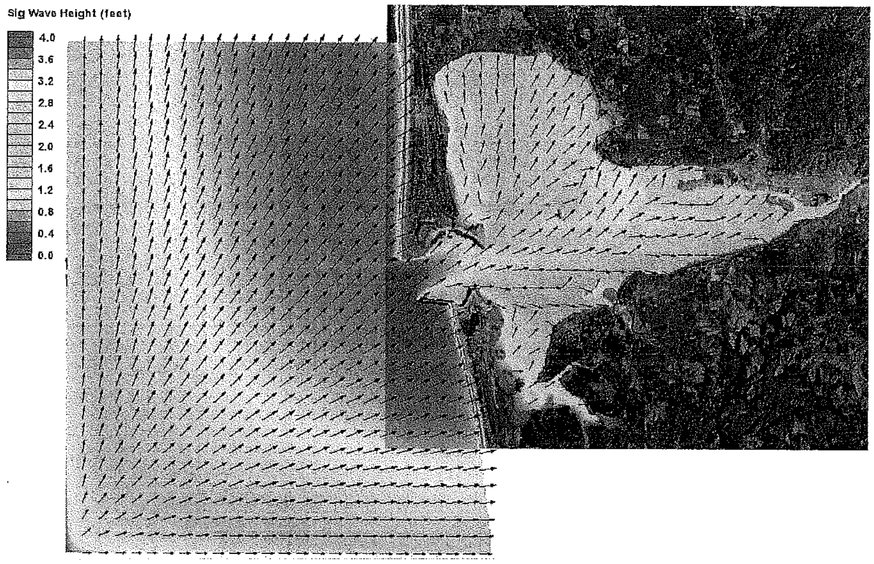

Figure 19. Example of wave modeling results, large wave modeling domain, 2-year return storm event, wind from 230°T at MHHW tide level

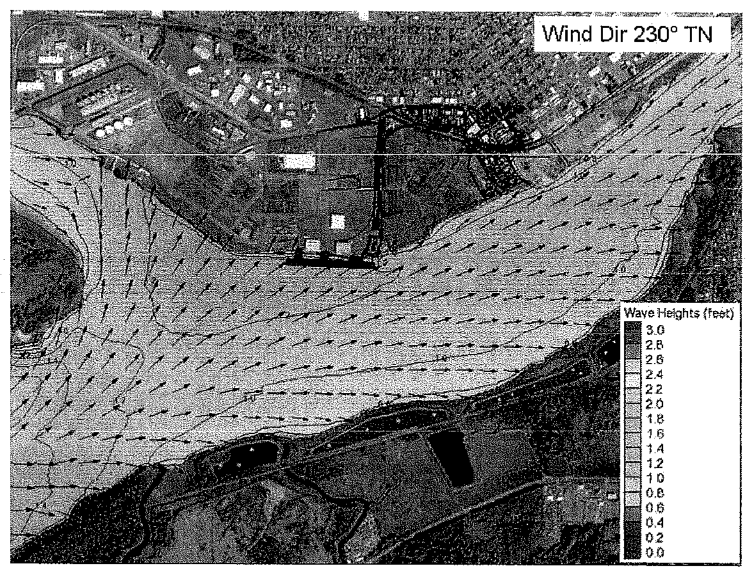

Figure 20. Example of wave modeling results, nested modeling domain, 2-year return storm event, wind from 230°T at MHHW tide level

The figures show snap-shots of significant wave heights over the modeling domains, in color format. Red color corresponds to higher wave heights, while blue color corresponds to smaller wave heights. To document wave parameters for existing conditions, 12 model gauges (reference points) were installed on the modeling domain. Figure 21 shows the locations of the reference points.

Figure 21. Location of reference points on the wave modeling nested domain

Wave parameters (wave height and period) were extracted at the reference points from the modeling results for each of the modeling storm events. Table 7 shows an example of wave modeling results at the 12 reference points for a storm approaching from the southwest (230^0). A complete set of documented wave parameters for all modeled wind speeds and directions is presented in Appendix A.

Table 7. Wave height and period at reference points for simulated storms from 230^0 direction

Point No.	Existing Conditions					
	Sig Wave Height (ft)			Wave Period Tp (s)		
	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr
1	1.3	2.0	2.2	3.0	3.3	3.3
2	1.2	1.9	2.1	2.7	2.7	2.7
3	1.2	1.9	2.1	2.7	2.7	3.0
4	1.1	1.6	1.8	2.7	3.3	3.3
5	1.0	1.6	1.7	2.2	2.7	2.7
6	1.0	1.5	1.7	2.2	2.7	2.7
7	1.2	1.8	2.0	2.7	3.0	3.0
8	1.0	1.6	1.7	2.2	2.7	2.7
9	1.0	1.6	1.7	2.2	2.7	2.7
10	1.1	1.8	2.0	2.7	3.0	3.0
11	1.1	1.8	1.9	2.7	3.0	3.0
12	1.1	1.7	1.9	2.5	3.0	3.0

3.3.2. Post-Project Conditions Impact Analysis

This section describes potential changes to existing wave conditions that may occur because of the proposed construction of the launch channel. The launch channel was built into the numerical modeling grid with dimensions discussed above (see Section 1), and wave modeling was repeated for the same modeling scenarios as for existing conditions. An example of existing and post-project wave modeling results for a 2-year return period storm event from the southwest is shown in Figure 22. Panel (a) for existing conditions and panel (b) for project conditions show wave modeling results for the 2-year return period and 230^0 direction storm event.

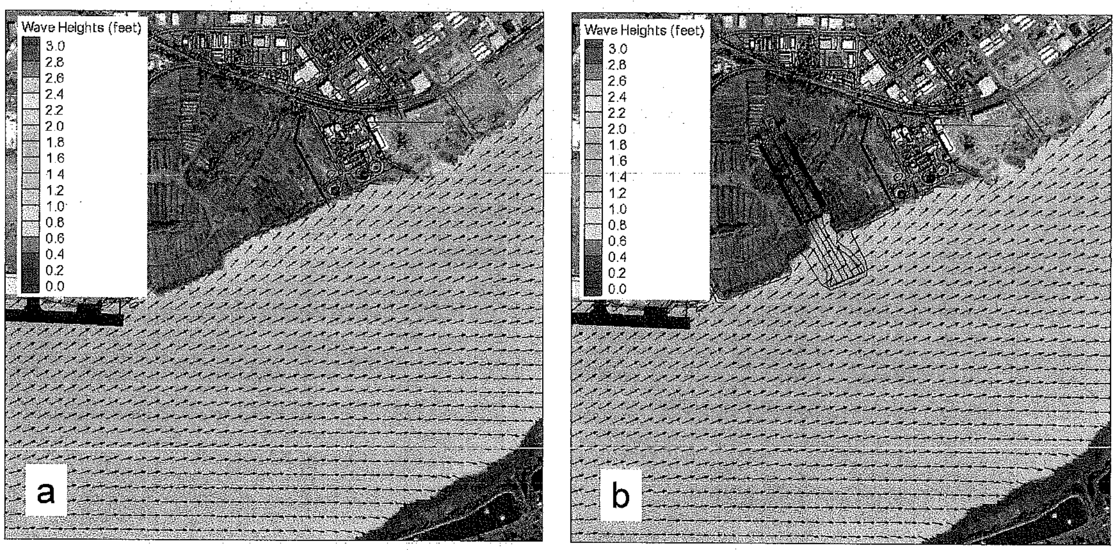

Figure 22. Significant wave height for (a) existing conditions, and (b) project conditions for 2-year return period event for waves from 230° T

Similar to the current velocities analysis, two methods were applied to detect and evaluate potential changes of wave parameters due to construction of the proposed launch channel. The first method (spatial) calculates the difference between significant wave heights for existing and project conditions at the same time in the simulation at each computation point in the model. An example of analysis with this method, plots of spatial difference of significant wave height, is shown in Figure 23.

In the figure red spectrum colors indicates an increase, blue color indicates a reduction, and white color indicates no change in wave heights between existing and project conditions. The figure shows a slight (insignificant) increase of wave heights in a localized area just outside of the submerged channel. This increase is due to the effect of wave steepening on channel slopes (shoaling effect)[7].

[7] Please note that the emerged channel is cut into the land. Therefore, red color inside of the emerged channel indicates an appearance of waves, not an increase of wave heights. These waves are small and confined between rock slopes and gates.

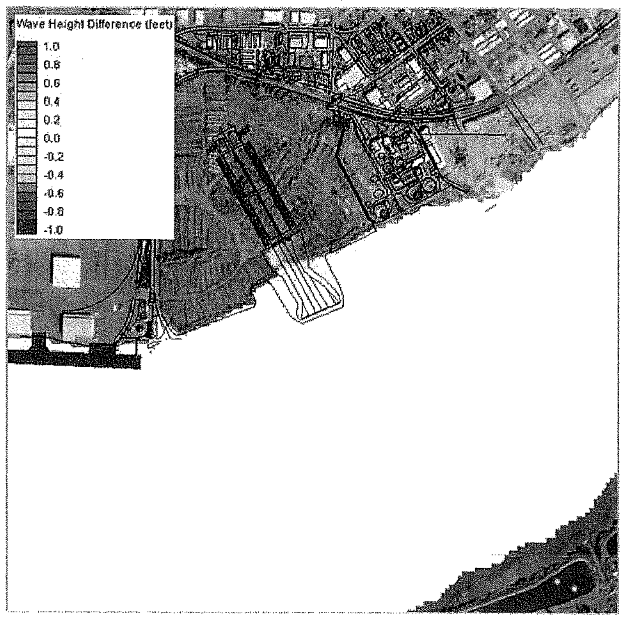

Figure 23. Significant wave height difference, 2-year storm event from SW (project minus existing conditions)

Outside of the launch channel footprint changes in wave heights are undetectable in the model and are expected to be negligible. The analysis of all other plots of differences showed a similar pattern: small changes of wave heights in the vicinity of the launch channel, and no change of wave heights outside of the project vicinity.

The second method of analysis (time domain) consisted of extracting wave heights from the reference stations (same stations that were used for documenting existing conditions) and comparing them to existing conditions. Table 8 is an example of wave height comparison between existing and project conditions. The table presents wave heights at reference stations for all modeled extreme storm events from 230° T, for existing and project conditions.

Table 8. Wave height and period at reference points for simulated storms at MHHW tide conditions for winds from 230° T

Point No.	Existing Conditions						Project Conditions					
	Sig Wave Height (ft)			Wave Period Tp (s)			Sig Wave Height (ft)			Wave Period Tp (s)		
	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr
1	1.3	2.0	2.2	3.0	3.3	3.3	1.3	2.0	2.2	3.0	3.3	3.3
2	1.2	1.9	2.1	2.7	2.7	2.7	1.2	1.9	2.1	2.7	2.7	2.7
3	1.2	1.9	2.1	2.7	2.7	3.0	1.2	1.9	2.1	2.7	2.7	3.0
4	1.1	1.6	1.8	2.7	3.3	3.3	1.0	1.6	1.8	2.7	2.7	3.3
5	1.0	1.6	1.7	2.2	2.7	2.7	1.0	1.6	1.7	2.2	2.7	2.7
6	1.0	1.5	1.7	2.2	2.7	2.7	1.0	1.5	1.6	2.7	3.0	3.0

Point No.	Existing Conditions						Project Conditions					
	Sig Wave Height (ft)			Wave Period Tp (s)			Sig Wave Height (ft)			Wave Period Tp (s)		
	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr
7	1.2	1.8	2.0	2.7	3.0	3.0	1.2	1.8	2.0	2.7	3.0	3.0
8	1.0	1.6	1.7	2.2	2.7	2.7	1.0	1.6	1.7	2.2	2.7	2.7
9	1.0	1.6	1.7	2.2	2.7	2.7	1.0	1.6	1.7	2.2	2.7	2.7
10	1.1	1.8	2.0	2.7	3.0	3.0	1.2	1.8	2.0	2.7	3.0	3.0
11	1.1	1.8	1.9	2.7	3.0	3.0	1.1	1.8	1.9	2.5	3.0	3.0
12	1.1	1.7	1.9	2.5	3.0	3.0	1.1	1.7	1.9	2.5	3.0	3.0

The following conclusions were derived from a comparison analysis of existing and project wave conditions from all storm events that were modeled:

- Significant wave height at the site is relatively small, generally less than 3 ft, even during the most extreme conditions.
- Changes to wave height are expected to be very small (0.1 to 0.2 ft), based on numerical modeling along the launch channel and adjacent areas.
- Small changes to wave height and patterns are expected primarily east of the launch channel, due to the predominant wind from the west.

4. Geomorphology - Existing and Project Conditions

4.1. Approach

Existing conditions of geomorphology at the project site were documented based on compilation, processing, review and analysis of the existing data, previous study results, and practical experience with coastal projects in Grays Harbor Estuary. Possible changes to geomorphic conditions due to the project were identified as a function of possible changes of hydraulic conditions (tides, currents, and waves described in Sections 2 and 3) and results of the sedimentation study for the proposed launch channel, as presented in Volume 1 of the report.

4.2. Geomorphology – Existing Conditions

The upper part of Grays Harbor Estuary is a relatively shallow area, characterized by extensive tidal flats (mudflats), submerged and exposed daily by tidal action. Figure 24 shows historic navigation charts in the project vicinity. The figure illustrates that the area of Cow Point was tidally inundated prior to 1928 and was subsequently filled in various stages to the present condition. Tidal flats fronting the project site appear to exist throughout all the available data, and are therefore persistent features.

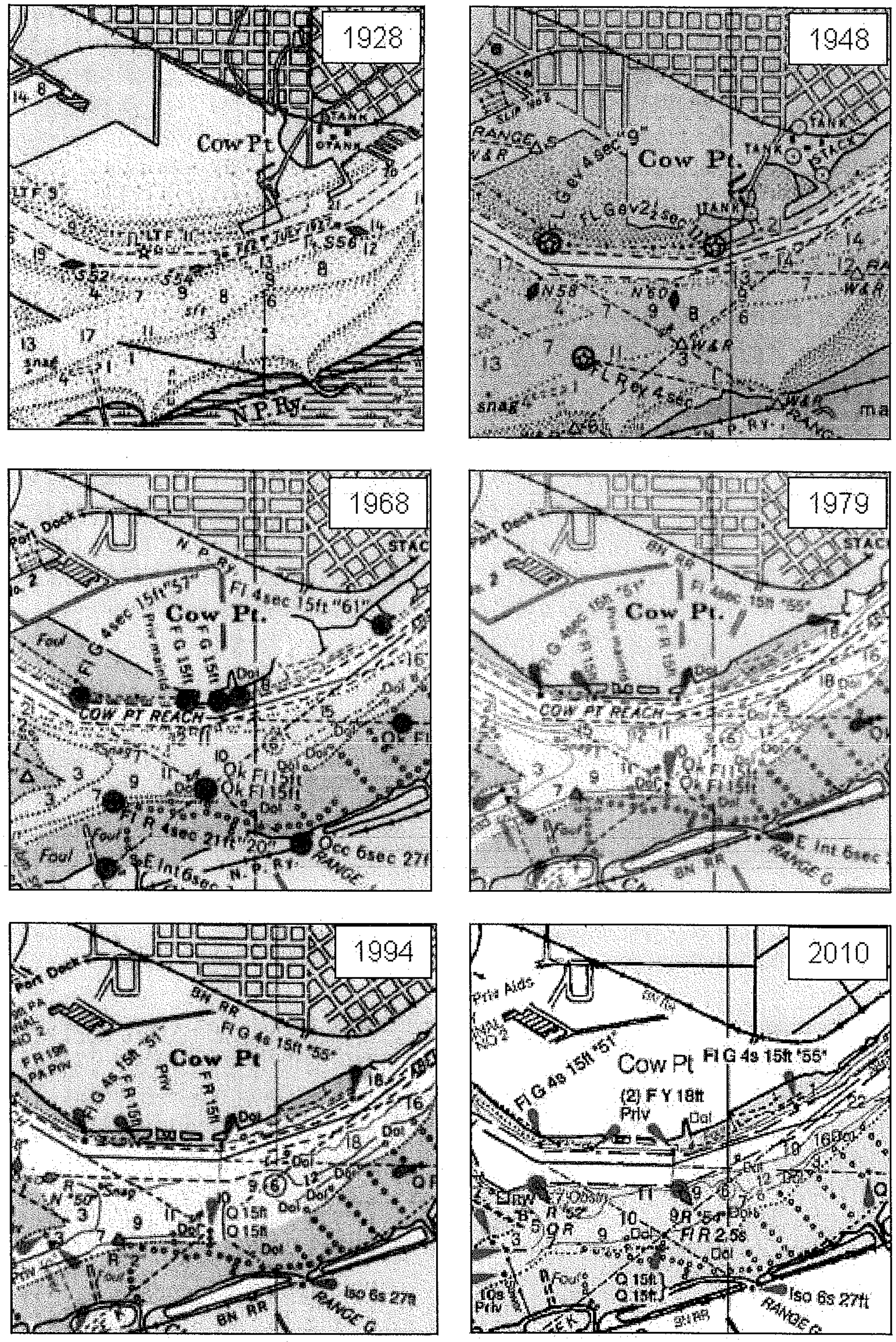

Figure 24. Historical navigation charts (NOAA) in the project area, date as noted

The proposed launch channel would be dredged/excavated through the 300- to 400-ft wide tidal flat. Figure 25, an oblique picture of the project site, shows a mudflat at the approximate location of the centerline of the proposed launch channel. The mudflat is bordered by the Federal Navigation Channel Aberdeen Reach at the seaward side, and by rock, gravel, rubble-composed berm at the landward side. The authorized depth in the Federal Navigation Channel Aberdeen Reach is -32 ft MLLW with 2 ft over-dredge, though the reach in front of the project site has not recently been dredged. The crest elevation on the berm at the landward side of the mudflat is at approximately +14 ft MLLW.

Mudflats at the project area are composed of soft silty material. Limited geotechnical data describes the upper 10 to 15 ft of sediment as very soft silt with traces of fine organics, fine sand, and scattered shell fragments. Typical moisture content of the samples ranged from 75 to 89 percent, with a plasticity index (PI) ranging from 35 to 47 percent. Reported blow count ranged from 0 to 1 blow per foot in the upper 15 ft. The grain size distribution taken 15 ft below the existing mudline indicates 97 percent of the sediment passed the #200 sieve (D50 = 0.009 mm). Samples collected by CH2MHill (2010) describe fine surface sediments, composed of 60% silt, 20% clay, and remainder sand. The sediments are therefore cohesive silts: threshold of motion for such cohesive materials may vary from 0.5 ft/s to 5 ft/s depending on consolidation and many other site specific factors requiring detailed investigation.

Analysis of existing geomorphologic trends at the project area was conducted based on compilation and processing of historical bathymetric survey data. For this purpose, annual condition surveys conducted by USACE were used. Though these surveys do not cover the shallow mudflats, the condition surveys extend sufficiently beyond the existing Federal Navigation Channel bottom-of-cut to detect morphological changes adjacent to the proposed launch channel.

Historical bathymetric survey data covering Cow Pt Reach and Aberdeen Reach areas were compiled from COE's database for the period from 2000 to the present time. Sequential post-dredge, pre-dredge, and channel conditions survey data were evaluated for analysis of sedimentation. These survey data were coupled to assure that no dredging events occurred between the dates of the survey. A total of 23 datasets of pre- and post-dredge surveys were identified. Sequential bathymetric survey data were compared, and depth differences were plotted to derive the sedimentation rate and patterns. Figure 26 is an example of bathymetric survey difference plots for six periods of time when no dredging work occurred between surveys. The figure includes the periods:

- 02/06/2002-10/04/2002 (approximate 8-month period)
- 02/07/2003-10/16/2003 (approximate 8-month period)
- 02/02/2004-09/27/2004 (approximate 8-month period)
- 02/18/2005-06/29/2005 (approximate 4-month period)
- 12/28/2005-01/23/2006 (approximate 1-month period)
- 01/03/2007-04/08/2008 (approximate 15-month period)

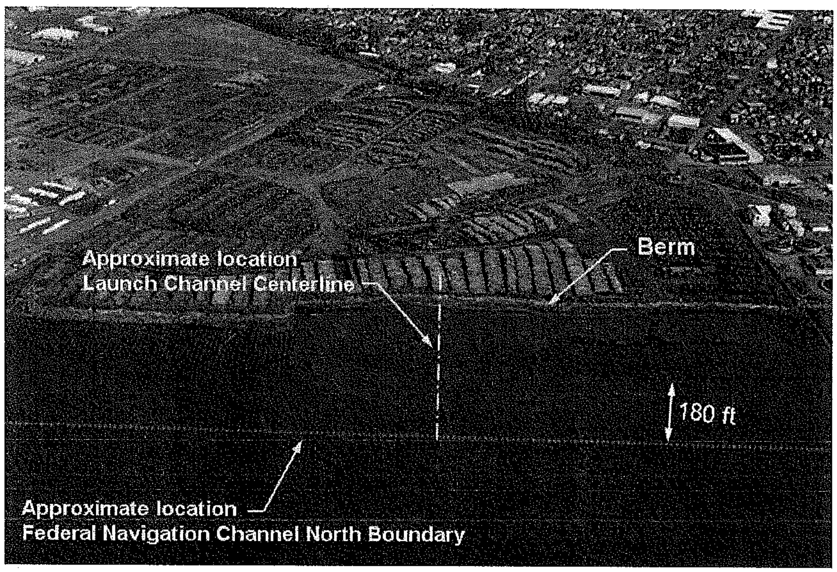

Figure 25. Annotated oblique photograph of project area from 1998 (photo: WA Department of Ecology)

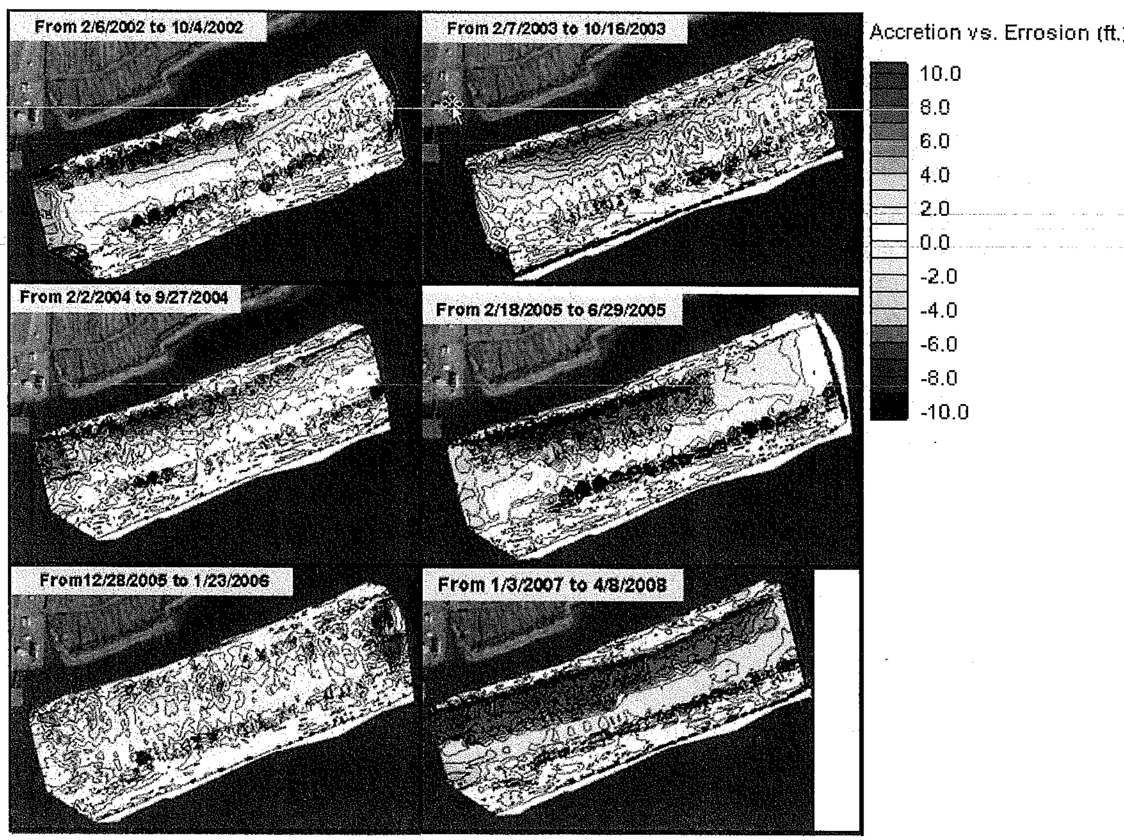

Figure 26. Example of bathymetric survey difference plots for periods indicated (red indicates accretion, blue indicates erosion)

Analysis of bathymetric survey data identified a trend of sedimentation at the north part of the Federal Navigation Channel adjacent to the mudflat. This sedimentation increases after maintenance dredging work and stabilizes at much smaller rates for five to six months thereafter. In absence of dredging, the trend of sediment deposition at the north side is very small, and probably would be detectable only on a scale of historical time (100 and more years).

4.3. Project Conditions Impact Analysis

Two possible impacts on existing geomorphologic conditions from construction of the launch channel are considered herein: direct and indirect. Direct impact may occur if construction of the launch channel would result in alteration of hydrodynamic conditions (waves and currents), which control sediment transport, deposition, and erosion. Non-direct impact may occur if construction of the launch channel would result in obstruction or extraction of littoral drift in the project area.

Direct impacts from in-water construction activities may occur, including but not limited to the following activities within 200 ft of the shoreline: dredging (mechanical and clamshell), in-water excavation, pile driving, grading and dressing of emergent and submerged slopes, placement of rock shoreline protection, and moorage of construction equipment.

Analysis and numerical modeling of currents and waves (see Sections 2 and 3) identified no changes or very minor changes in current velocities and wave parameters directly and at close proximity to the launch channel. No changes in existing hydrodynamic conditions (waves and currents) were determined at a distance 1,300 ft and further from the launch channel, in all directions. Based on the modeling results, it appears that no direct impact on existing geomorphologic conditions will occur outside of the constructed launch channel area; specifically, outside the boundaries of the launch channel.

Indirect impact on existing geomorphologic conditions will be determined by the type and amount of sediment accumulated in the channel and the frequency of maintenance dredging and practice for dredged material disposal. The detailed launch channel sedimentation analysis, presented in Volume 1 of the report, concludes that approximately 32,400 CY of sediment may accumulate per year in the launch channel, and required maintenance dredging would remove approximately 17,100 CY of sediment every six months. This implies that during the lifetime of the project (assume five years) the anticipated maximum amount of sediment to be dredged is approximately 162,000 CY. For comparison, the adjacent Aberdeen and Cow Pt. Reaches of the Federal Navigation Channel have required maintenance dredging of about 725,000 CY each year. Therefore, this volume represents a small fraction of dredging activities in the adjacent Federal Navigation Channel.

As discussed above, most of the sediment deposited on the existing mudflats that is expected to deposit in the launch channel would be fine silt material delivered by the Chehalis River. If this fine silty sediment were not deposited in the launch channel or

Federal Navigation Channel, it would predominately remain in suspension while transiting through Grays Harbor Estuary. This sediment could temporarily settle in the estuary, but could easily be re-suspended by small waves and currents transporting these fine silts further toward the open ocean. Therefore, only a small fraction of sediment that may potentially deposit in the launch channel would contribute to the geomorphologic processes and trends.

Considering the limited area of possible alteration in existing hydrodynamic conditions, the type and relatively small volume of sediment involved in sedimentation, and short lifetime of the project, it is concluded that no significant direct or indirect impacts to the existing geomorphic conditions should be expected from construction of the launch channel.

5. References

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Appendix A

Modeled Wind Speeds and Directions

APPENDIX A

WAVE HEIGHT AND PERIOD AT REFERENCE POINTS FOR SIMULATED STORMS (2-, 25- AND 50-YEAR RETURN PERIOD) FROM 230^0, 250^0 AND 270^0 DIRECTION TRUE NORTH

	230 °						250 °						270 °					
	Sig Wave Height (ft)			Wave Period Tp (s)			Sig Wave Height (ft)			Wave Period Tp (s)			Sig Wave Height (ft)			Wave Period Tp (s)		
Point No.	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr	2-yr	25-yr	50-yr
1	1.3	2.0	2.2	3.0	3.3	3.3	1.3	1.8	1.9	3.0	3.0	3.0	1.2	1.6	1.7	2.4	3.0	3.0
2	1.2	1.9	2.1	2.7	2.7	2.7	1.2	1.7	1.8	2.6	2.7	2.7	1.1	1.5	1.6	2.4	2.7	2.7
3	1.2	1.9	2.1	2.7	2.7	3.0	1.2	1.7	1.8	2.5	2.7	3.0	1.1	1.4	1.5	2.4	2.7	2.7
4	1.1	1.6	1.8	2.7	3.3	3.3	1.0	1.3	1.4	3.0	3.0	3.3	0.8	1.1	1.1	2.7	3.0	3.0
5	1.0	1.6	1.7	2.2	2.7	2.7	0.9	1.2	1.3	3.0	3.0	3.0	0.8	1.0	1.0	2.7	3.0	3.0
6	1.0	1.5	1.7	2.2	2.7	2.7	0.9	1.2	1.3	2.7	2.7	2.7	0.7	1.0	1.0	2.7	3.0	3.0
7	1.2	1.8	2.0	2.7	3.0	3.0	1.1	1.6	1.7	3.0	3.0	3.0	1.0	1.3	1.4	2.7	2.7	2.7
8	1.0	1.6	1.7	2.2	2.7	2.7	0.9	1.3	1.3	2.5	2.7	2.7	0.8	1.0	1.1	2.7	2.7	2.7
9	1.0	1.6	1.7	2.2	2.7	2.7	0.9	1.2	1.3	2.7	2.7	2.7	0.7	1.0	1.0	2.5	2.7	2.7
10	1.1	1.8	2.0	2.7	3.0	3.0	1.1	1.5	1.6	3.0	3.0	3.3	1.0	1.3	1.4	2.7	3.0	3.0
11	1.1	1.8	1.9	2.7	3.0	3.0	1.0	1.4	1.5	2.8	3.0	3.3	0.9	1.2	1.3	2.7	3.0	3.0
12	1.1	1.7	1.9	2.5	3.0	3.0	1.0	1.4	1.5	2.4	2.7	3.0	0.8	1.1	1.2	2.7	3.0	3.0

TECHNICAL REPORT VOLUME 3
SR520 COASTAL ENGINEERING REPORT
LAUNCH CHANNEL ARMOR SIZE AND DOLPHIN SCOUR PROTECTION DESIGN

This document was prepared by a Professional Engineer.

VLADIMIR SHEPSIS
STATE OF WASHINGTON
34455
REGISTERED
PROFESSIONAL ENGINEER

Vladimir Shepsis, Ph.D., P.E.
Principal, Coast and Harbor Engineering

COAST & HARBOR
ENGINEERING

110 Main Street, Suite 103
Edmonds, WA 98020
Ph 425 778.6733
Fax 425 977.7416

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COAST & HARBOR
ENGINEERING

Technical Report Volume 3

SR520 Coastal Engineering Report
Launch Channel Armor Size
and Dolphin Scour Protection Design

Executive Summary

This report is Volume 3 of a three-volume Technical Report prepared by Coast & Harbor Engineering (CHE) under Task Orders 1 & 2 for the KIEWIT-HNTB SR 520 Pontoon Construction Design-Build Project and presents the results of work completed under the Scope of Work, Task 3, Launch Channel Side Slope Rock Dimensions and Task 6, Dolphins Scour Analysis and Protection Design. Volume 1 of the Technical Report presents the results of analysis and numerical modeling of wave and current hydrodynamics of the project area. Volume 2 of the Technical Report presents the results and analysis of coastal hydraulic and geomorphologic conditions.

The objective of this report is to determine the minimum rock size that would be stable on launch channel side slopes (3H:1V slope) under impact from propeller wash by tugs operating in the launch channel. This report also determines the depth of possible propeller-induced scour at the two turning dolphins and dolphins supporting the waler beams in the launch channel. The need for scour protection measures at these structures was also evaluated.

Launch channel armor rock and scour at the dolphins analysis was conducted using a two-dimensional JETWASH model. This model has been successfully used for numerous propwash scour studies in the State of Washington and other regions of the U.S. and overseas. JETWASH was accepted by EPA Region 8 and U.S. Army Corps of Engineers for analysis and design of armor cap material for isolating contaminated sediment.

Input parameters for propwash modeling include the type and dimensions of the tugboat, dimensions and operational characteristics of the propulsion system, the position of the propulsion system relative to the channel slope, bathymetry of the channel bottom and slope, and tide elevation (or water depth). These parameters were determined and coordinated with the Project Team. The tug Daniel Foss was selected as a design vessel for propwash modeling and analysis.

For simplicity of design and construction, a single rock gradation for armoring the channel slopes is recommended. This size of the rock is determined for the worst case scenario, namely maximum flow velocity produced by propwash. The estimated median armor stone diameter (D50) is approximately 1.5 ft with an equivalent median armor stone weight, W50, of approximately 375 lbs.

Estimated depths of scour at the mudline of piles exposed to propwash are listed in Table 4, and range from 1.7 ft to 4.0 ft below the designed channel bottom surface. Rock that would protect the bottom sediment from scour at the piles was calculated to be in the range between 0.5 ft and 0.8 ft in diameter.

The design recommendations provided herein are preliminary and should be optimized as tug operational parameters are further developed and the project progresses toward final design.

COAST & HARBOR
ENGINEERING

Technical Report Volume 3

SR520 Coastal Engineering Report Launch Channel Armor Size and Dolphin Scour Protection Design

1. Introduction

This report is Volume 3 of a three-volume Technical Report prepared by Coast & Harbor Engineering, Inc. (CHE) under Task Orders 1 & 2 for the KIEWIT-HNTB SR 520 Pontoon Construction D-B Project. Volume 1 of the Technical Report presents the results of analysis and numerical modeling conducted for Task 1 of the Scope of Work, Channel Sedimentation Study. Volume 2 of the Technical Report presents the results and analysis for Task 2, Grays Harbor Site Coastal Hydraulic and Geomorphologic Conditions.

This current Technical Report, Volume 3, describes analysis and provides recommendations for rock size to armor side slopes of the launch channel and to prevent scour at the two turning dolphins and dolphins supporting the waler beams in the launch channel (also referenced as channel waler dolphins).

The geometry of the launch channel, including the side slopes (3H:1V slope) and location of turning and waler dolphins, was provided to CHE in drawings by HNTB. Figure 1 shows the launch channel plan provided by HNTB. Typical cross-sections of the launch channel considered herein are shown in Figure 2. Subsequent analysis of channel armoring and scour protection is based upon the HNTB provided design as depicted in Figures 1, 2, and 3. Stability of side slope armor rock, scour at the turning dolphins and piles, and scour at supporting waler beams were examined with regard to hydrodynamic forces generated by propwash from the design tugboats to be operated in the launch channel. Scouring effects from wind waves and river and tidal flows are considered insignificant (See Volume 2 of the report), and are not taken into account in this study.

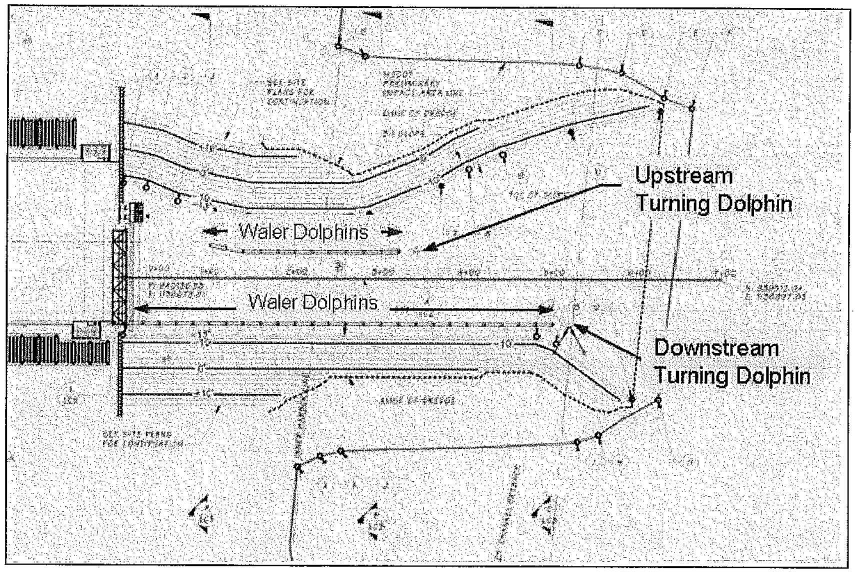

Figure 1. Plan view of the proposed launch channel (drawings provided by HNTB)

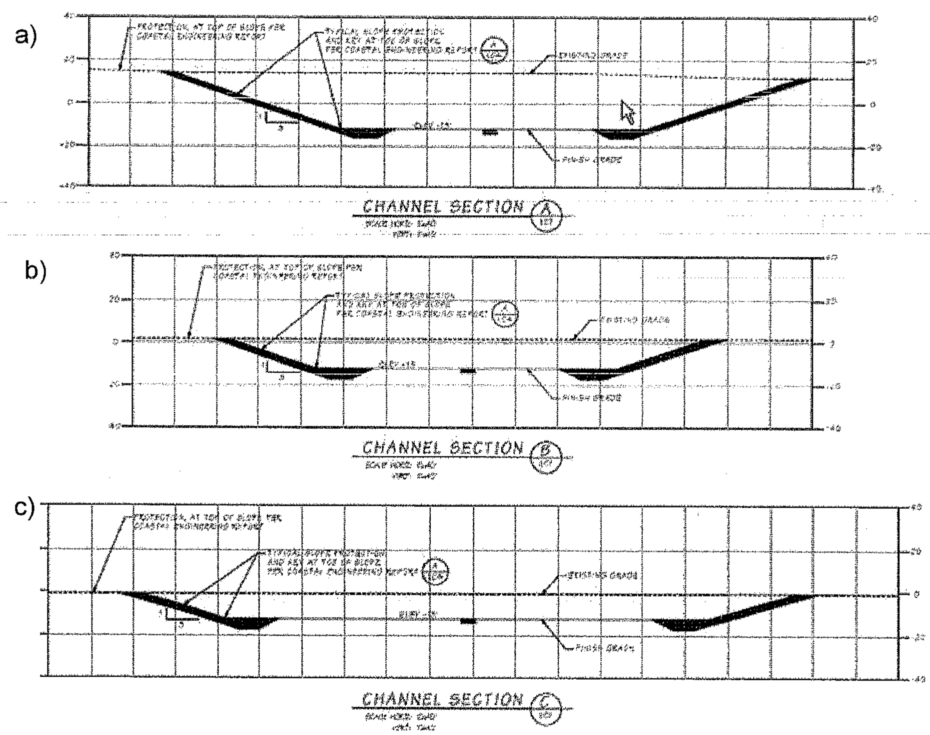

Figure 2. Cross-sections of original launch channel design (a) landward, (b) middle, and (c) seaward parts (drawings provided by HNTB)

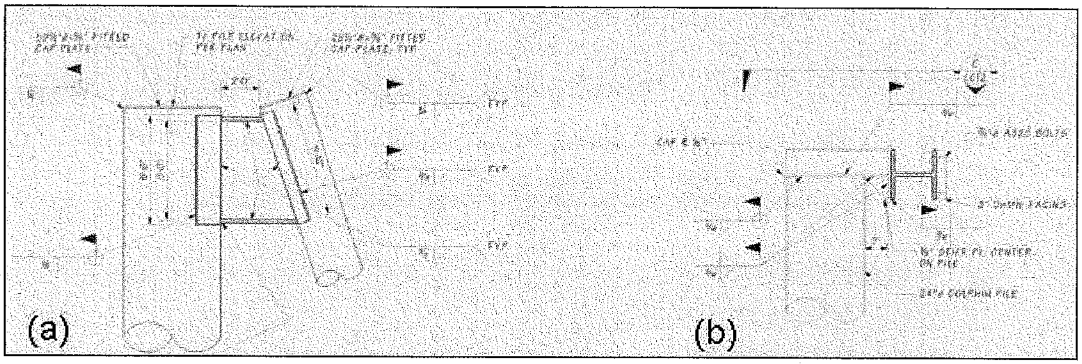

Figure 3. Elevation detail of (a) turning dolphin and (b) channel waler dolphin (drawings provided by HNTB)

2. Launch Channel Side Slope Rock Dimensions

2.1. Approach

Launch channel slope stability analysis was conducted using the two-dimensional JETWASH model (V. Shepsis and D. Simpson, 2001). The numerical model simulates the flow field behind a rotating propeller and the velocity interaction with the bottom. The JETWASH model has been successfully used for numerous propwash scour studies in the State of Washington and other regions of the U.S. and overseas. JETWASH was accepted by EPA Region 8 and U.S. Army Corps of Engineers for analysis and design of armor cap material for isolating contaminated sediment.

The JETWASH model was calibrated and verified previously for various types and operational parameters of propulsion systems. No specific model validation and calibration of JETWASH was anticipated under the current study.

Armor rock dimensions were computed using multiple well-known and practical engineering formulations. The recommended armor rock sizes were selected after comparing the computed results and utilizing previous CHE experience with similar projects.

2.2. Input Parameters

Input parameters for propwash modeling included the type and dimensions of the tugboat, dimensions and operational characteristics of the propulsion system, the position of the propulsion system relative to the channel slope, bathymetry of the channel bottom and slope, and tide elevation (or water depth). Major input parameters that were used in the analysis are described below. The tug Daniel Foss was selected as a design vessel for analysis of rock stability[1].

[1] This tugboat was suggested by Kiewit Infrastructure West (Kiewit) by email dated July 29, 2010 and was confirmed at a meeting held on September 14, 2010.

The Daniel Foss (design tug) is powered by twin azimuth stern drive propellers. Tug dimensions and operational parameters to be used for propwash modeling were provided to CHE by Kiewit. The main provided parameters are depicted in Table 1.

Table 1. Propeller wash input parameters and simulation cases

Parameter	Case 1	Case 2	Case 3
Water Depth (ft)	19	19	19
Propeller Draft (ft)	7.3	7.3	7.3
Ducted Propeller Diameter (ft)	6.0	6.0	6.0
Applied Power/engine (HP)	413	825	1,237
Applied Power/engine (% full)	25%	50%	75%
Thrust Developed (lbs)	11,250	22,500	33,750

Power delivered to the propeller shaft is a critical parameter for computing propwash velocity. This power may vary throughout towing operation, depending on the location of the towed pontoon, and conditions of wind, waves, and flow velocities. Based on information from Kiewit and coordination with the Project Team, the area of the launch channel was divided into three zones of possible maximum applied power. These zones are shown in Figure 4 in color format. Modeling cases referenced in Table 1 above correspond to these three zones.

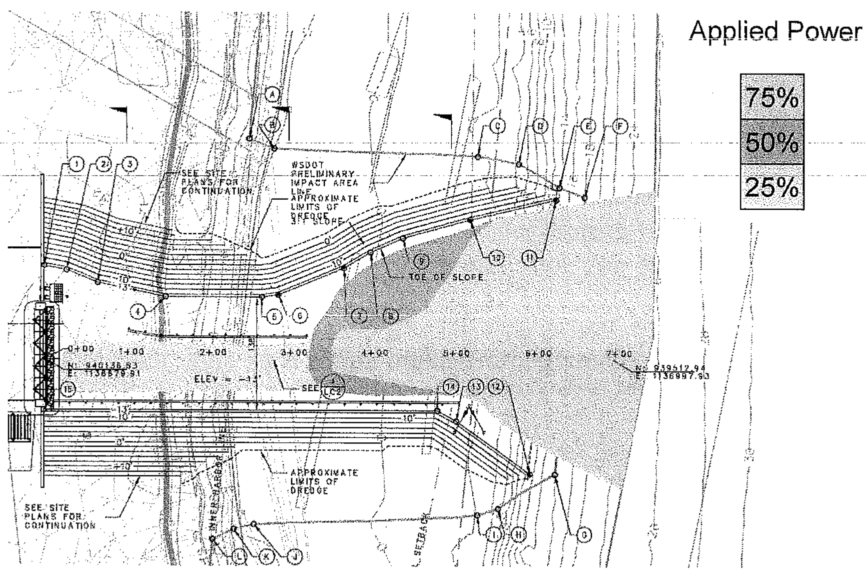

Figure 4. Zones of applied maximum tug power by location within the launch channel

The other critical parameter for propwash modeling includes the position and orientation of the tugboat relative to the channel, which controls angle of slope exposure to the propeller jet. The design position of the tug was selected to represent a worst case scenario. The side of the tugboat was aligned against the waler.

Three angles of propeller axis relative to the toe of slope were modeled: 0° (parallel to toe of slope), 15°, and 30° off alignment to the toe of slope. This assumption was coordinated with the Project Team and confirmed during a meeting on September 14, 2010. The potential blockage of the propeller wash by the pontoon itself was not considered directly during modeling for armor stability or bottom scour.

2.3. Propeller Wash Modeling and Results

Once input parameters were established and coordinated with the Project Team, modeling was conducted for all design cases. Modeling results were presented as a plan view of bottom velocities behind the propeller, and as bottom velocities along the toe of the upstream (north) and downstream (south) channel slopes. The area near the toe of the slope experiences the highest propwash velocity because of the least lateral distance from the propeller location to the side slope. This velocity (at the toe) is used further as a design criterion for analysis of armor rock stability.

An example of JETWASH modeling results, showing a plan view velocity pattern near the bottom, is illustrated in Figure 5 for Case 3 (75 percent power) with propeller axis orientated parallel to the toe of slope (note Figure 5 rotated 180 deg from Figure 4). Figure 6 shows modeling results of near-bottom velocities along the toes of channel slopes (both upstream and downstream slopes) for the same case. Please note that each velocity profile along the toe of slope in Figure 6 is offset laterally, but parallel to, the axis of the velocity jet. For all modeling cases near-bottom velocity was calculated at a height of 0.85 ft above the bottom.

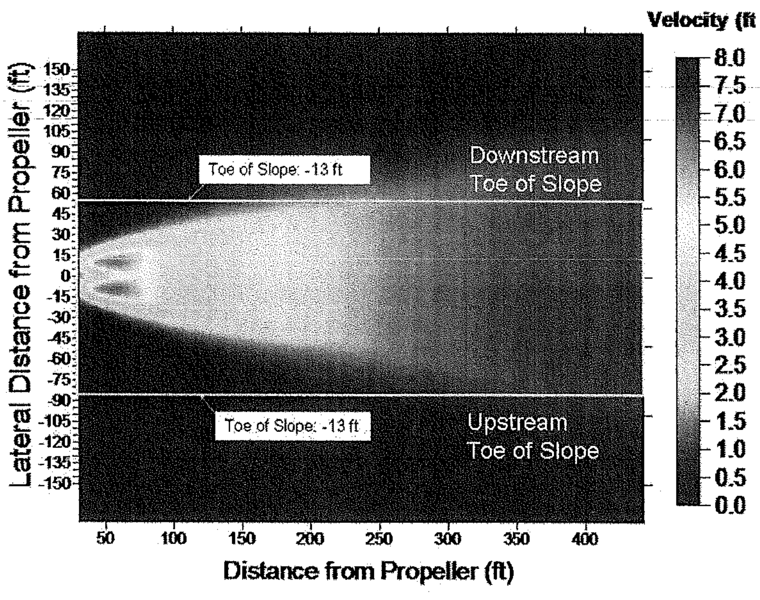

Figure 5. Plan view of near-bottom velocity along the launch channel, 75% applied power with propeller axis parallel to the toe of slope

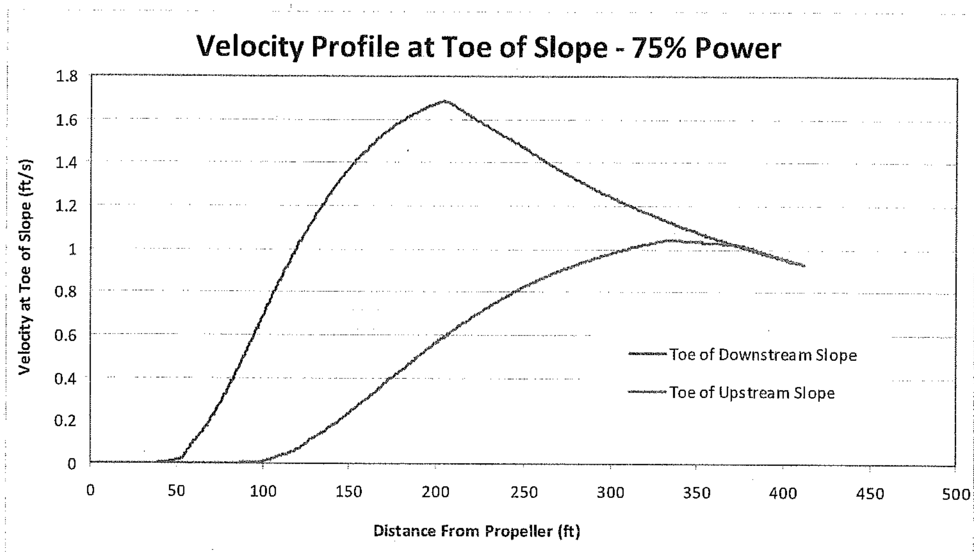

Figure 6. Near-bottom velocity along the toe of slope for 75% applied power with propeller axis parallel to the toe of slope

From Figures 5 and 6, it can be seen that velocity decreases with horizontal distance from the propeller and laterally from the propeller jet axis. Relatively high velocity is experienced at the bottom directly below the axis of the propeller jet. The velocity at the toe of the launch channel side slope is reduced (less than 2 ft/s for the Case 3 and offset from the jet axis).

The propulsion units on the Daniel Foss are independent and can turn to direct thrust in any direction in a near-horizontal plane. For that reason, it was assumed that velocity could be directed across slope in all zones of applied power shown in Figure 4. Propulsion parameters were input to the JETWASH model and near-bottom velocity was calculated accounting for the flat channel bottom between the location of the propeller and the toe of slope, and the 3H:1V channel side slope. Results of calculated propwash-induced flow velocities directed across slope are summarized in Table 2.

Table 2. Across-slope bottom velocities for simulation cases

Maximum Bottom Velocity ft/sec		
Case 1 (25% Power)	Case 2 (50% Power)	Case 3 (75% Power)
5.3	7.5	9.2

2.4. Armor Rock Recommendations

Armor rock dimensions for the launch channel slope protections were computed using several standard engineering methods, including those described in PIANC Sup. No 57 (1987), Schiereck (2001), FHWA (1997), PIANC (1992), and Prosser

(1986). Engineering judgment was applied to the range of reasonable results to select the recommended rock size. The selected rock size falls in the upper range of the results, between the mean and maximum computed size. For simplicity of design and construction we recommend using a single gradation of rock for armoring the channel slopes. This median size of the rock that characterizes the gradation is determined for the worst case scenario, namely, maximum flow velocity produced by the propwash. The maximum propeller-induced flow velocity modeled for the channel slope was for modeling Case 3, shown in Table 2.

For this Case 3 propwash scenario (velocity of 9.2 ft/sec) the resulting required median armor stone diameter, D_{50}, is approximately 1.5 ft, with an equivalent median armor stone weight, W_{50}, of approximately 375 lbs for angular rock having a density of 170 lb/ft^3.

The design recommendations provided herein are preliminary and should be optimized as tug operational parameters are further developed and the project progresses towards final design.

3. Dolphins Scour Analysis and Protection

3.1. Approach

Similar to the analysis of the launch channel side slope, scour analysis at the dolphins was conducted using the two-dimensional numerical model JETWASH. The numerical model simulated the flow field behind the propeller along the channel bottom at the turning dolphins and channel waler dolphins.

The computed propwash velocities near the dolphins were compared to threshold velocities of bottom sediment. Where propwash velocity exceeded threshold velocity of bottom sediment, the depth of scour was determined using empirical formulations. Scour protection is recommended where bottom scour was shown to occur, assuming that depths of scour at the turning and waler dolphins are not acceptable for structural stability

3.2. Input Parameters

Input parameters for scour analysis that includes propwash (See Section 2.2) includes bottom sediment characteristics at the turning and waler dolphins. These sediment characteristics were determined from soil borings data that have been collected in the vicinity of the launch channel. The sediments at the bottom of the proposed launch channel (elevation -13.0 ft MLLW) are dominated by silts and clay (80 to 95 percent by weight) and minor amounts of fine sand (5 to 15 percent by weight)[2]. The median size selected to represent sediment at the constructed channel bottom is 0.01 mm. Nearly all the sampled sediment is smaller than 0.1 mm.

[2] More detailed description of bottom and slope sediments are described in Volumes 1 and 2 of this Report.

3.3. Propwash Velocity and Scour Analysis

Bottom velocities generated by tug propeller wash at the dolphins were determined from numerical modeling of propwash generated by the design tugboat (see Section 2 of this report). Maximum bottom velocities for modeling Cases 2 and 3 were extracted at the turning and waler dolphins. For this purpose, the tugboat was assumed to create the maximum bottom velocity at the location of the pile or dolphin. Figure 7 is an example of bottom velocity as a function of distance from the propeller, and the velocity peak is assumed to be at a turning dolphin location.

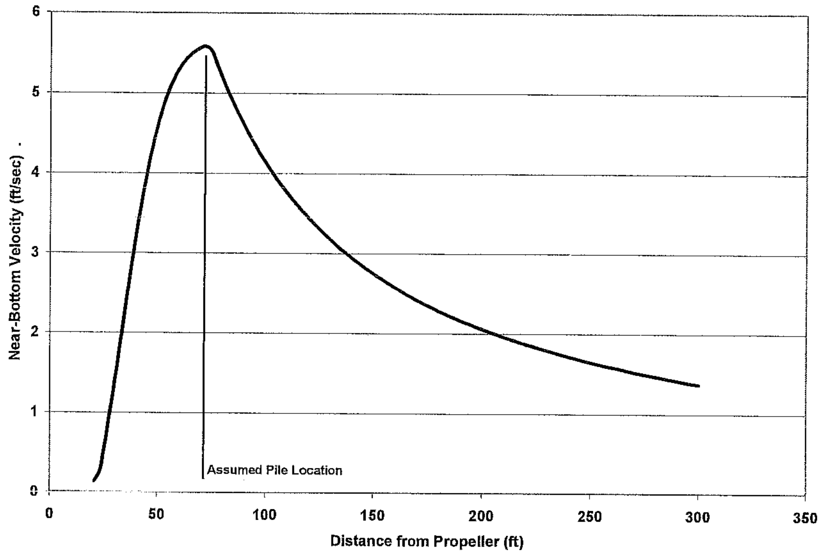

Figure 7. Near-bottom velocity below axis of propeller jet

Table 3 summarizes maximum bottom velocities that may occur during tugboat operations at dolphins in zones of 50 and 75 percent power[3]. The estimated threshold velocity of non-cohesive bottom sediment of size 0.01 mm is less than 1 ft/sec, and of size 0.1 mm is approximately equal to 1 ft/sec.

The table shows that maximum propwash bottom velocities significantly exceed threshold velocity of bottom sediment at all dolphin locations. Therefore, scour at the turning and waler dolphins would occur when a tugboat operates at design conditions in the launch channel.

[3] Scour estimates for the zone of 50 percent power were assumed to apply to the zone of 25 percent power as a means of accounting for uncertainty of extreme conditions and the risk to the rigid structural element (pile and waler system).

Table 3. Maximum propwash velocities at channel bottom

Maximum Near-Bottom Velocity (ft/sec)	
Zone of 50 Percent Power	Zone of 75 Percent Power
4.6	5.6

Reliable methods to accurately predict depth of propwash scour of fine cohesive sediment are not available. The estimate of scour depth is also complicated by the uncertainties in frequency of propwash occurrence, the duration of each occurrence in the vicinity of the dolphins, and the rate of infill following each scour event. Two empirical methods were used to estimate a potential depth of scour at the dolphins: Hamill (1988) and Whitehouse (1998). The first is a semi-empirical method that was developed for a flat bottom composed of non-cohesive sediment impacted by a propeller jet. The second is a simplified empirical method based on laboratory and field case studies at circular piles affected by steady flow (river currents)[4]. The final depth of scour was estimated considering results of the two methods as endpoints of a possible range of scour.

Table 4 summarizes the estimated scour depth at the dolphins in the launch channel computed with the approach described above.

Table 4. Estimated equilibrium scour depth at dolphins

		Scour Depth (ft)	
Dolphin Type	Pile Diameter (ft)	Zone of 50% Power	Zone of 75% Power
Turning Dolphin	5	3.6	4.0
Batter Pile	2.5	2.0	2.4
Waler Dolphin	2.0	1.7	2.0

3.4. Dolphin Scour Protection Recommendations

This section is prepared assuming that depths of scour at the turning and waler dolphins are not acceptable for structural stability, and scour protection would be required at all dolphins. Using maximum bottom velocities at each dolphin (Table 3) a size of stable scour protection rock material was computed with two different engineering methods, Ishbash and USBR using the Riprap Design System (West Consultants 2005). The recommended size of stable material is the average of the

[4] This method predicts a possible scour depth, S_e, as a function of pile diameter in the ranges from approximately 1.3D to 2.0D, where D is the pile diameter. Considering infrequent propwash events (for the launch channel) versus steady flow (used to develop the empirical method) the analysis uses the relationship, S_e =1.3D.

above methods. Table 5 presents the recommended stable size of rock material to prevent scour at each of the dolphins.

Table 5. Recommended size of scour protection rock for piles

Median Rock Size (ft)	
Zone of 50 Percent Power	Zone of 75 Percent Power
0.5	0.8

Alternatively, considering the predicted sedimentation in the channel (See Volume 1 of the report) the construction of scour protection can be postponed. A monitoring program is recommended for the first 6 to 12 months of operation in the launch channel. The monitoring program will validate the predicted depth of scour and determine if scour holes would be partially silted due to sedimentation between scouring events.

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This document was prepared by a Professional Engineer.

VLADIMIR SHEPSIS
STATE OF WASHINGTON
34455
REGISTERED
PROFESSIONAL ENGINEER

Vladimir Shepsis, Ph.D., P.E.
Principal, Coast and Harbor Engineering

COAST & HARBOR
ENGINEERING

110 Main Street, Suite 103
Edmonds, WA 98020
Ph 425 778.6733
Fax 425 977.7416

Technical Memorandum - DRAFT

SR520 Pontoon Construction Design-Build Project Addendum to Coastal Engineering Report, Vol. 1 - 3

1. Introduction

Coast & Harbor Engineering (CHE) has prepared this technical memorandum upon request from KIEWIT-HNTB SR 520 Pontoon Construction Design-Build Project as an addendum to the previously prepared SR520 Coastal Engineering Report (CER), Volumes 1 - 3. The objective of this memo is to identify possible changes to findings and conclusions from the SR520 CER due to modification of the initial launch channel configuration. Initially (as analyzed in the CER) the launch channel was designed with uniform channel slopes at 1V:3H. In accordance with the initial design, the channel slopes were armored with rock on each side through the entire approximately 620 ft channel length. Figure 1 shows initial plan view and cross sections of the launch channel used to generate the SR520 CER.

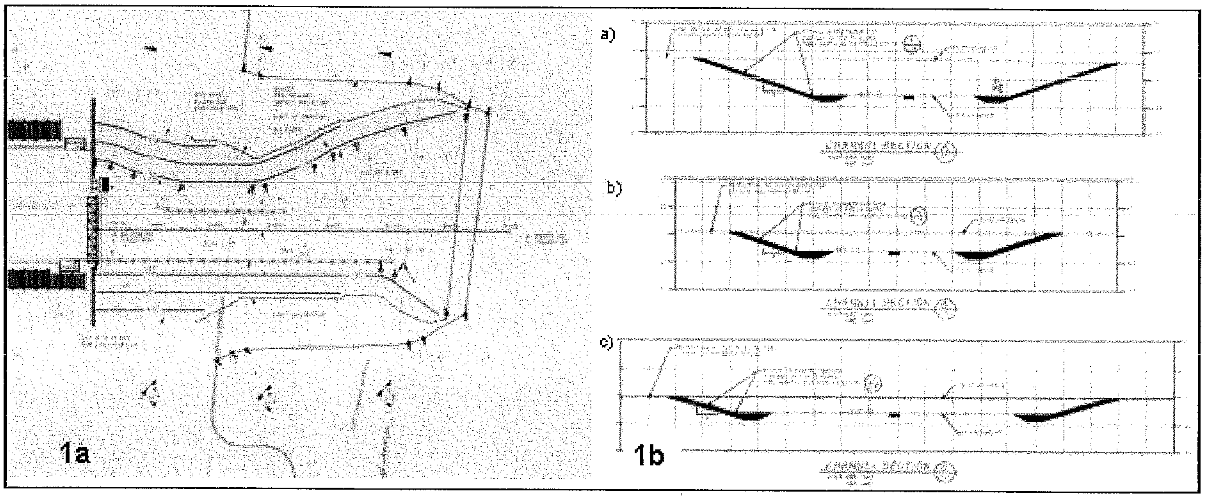

Figure 1 - Plan view (1a) and cross sections (1b) of the initial launch channel design.

The launch channel cross sections were subsequently reconfigured upon WSDOT request to reduce the extent of armoring on the slope. The rock armor upon reconfiguration is proposed only along a portion of the emergent channel. Channel slopes in the immediate vicinity of the gates remain at 1V:3H and transition to 1V:5H at the terminus of rock armor. Total length of the reconfigured channel armoring along the slope is less than 90 ft (instead of 620 ft) on each side of the channel. Figures 2 and 3 show the plan view and cross section details for the reconfigured launch channel.

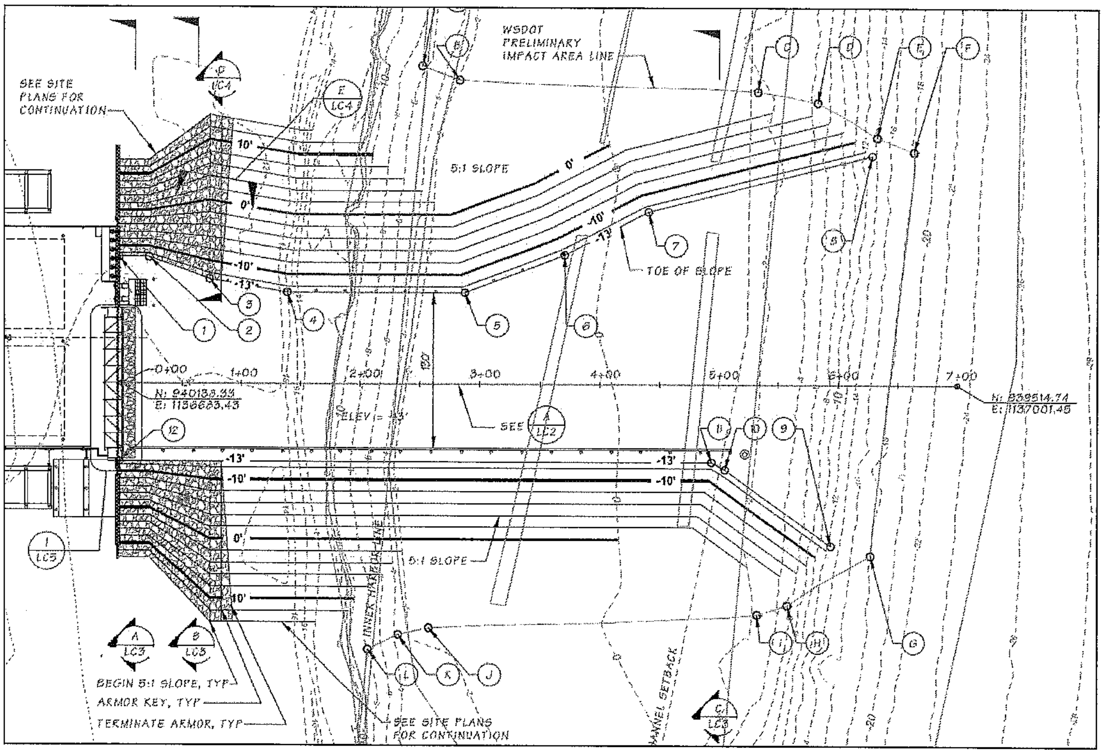

Figure 2 - Reconfigured launch channel plan (provided by HNTB 1/20/2011)

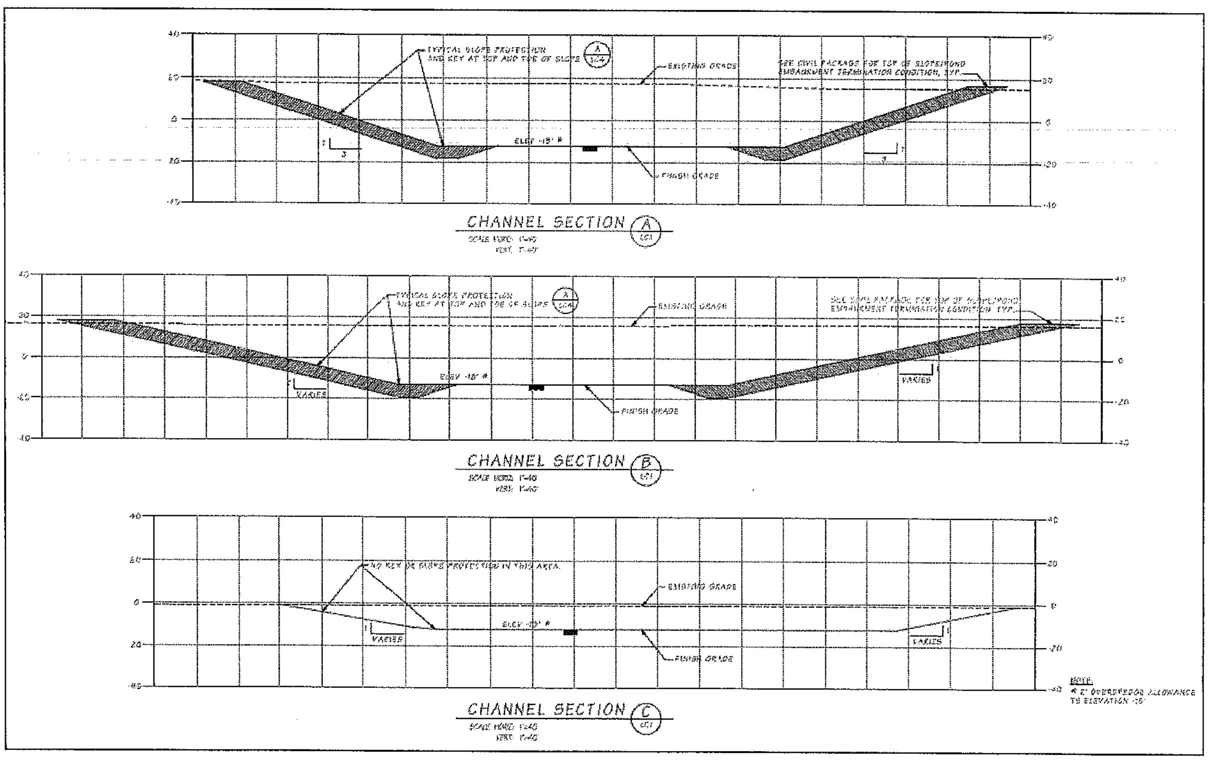

Figure 3 - Reconfigured launch channel sections (provided by HNTB 1/20/2011)

Table 1 compares key elements of the initial and reconfigured launch channel. These were evaluated to determine if the previous analysis, recommendations, and conclusions require revisions for the reconfigured channel, compared to the initial channel design. Overall, the conclusions and recommendations presented in the CER remain valid for the reconfigured launch channel. Differences and revisions to each volume of the Coastal Engineering Report for the reconfigured launch channel are discussed below.

Table 1 - Comparison of key project elements for the initial and reconfigured channel and potential revisions to Coastal Engineering Report.

Element	Initial Channel	Reconfigured Channel	Potential Revisions
Armor Extent	Emergent and submerged channel to be fully armored.	Emergent channel to be partially armored.	Sedimentation Hydraulics Geomorphology Armor Size & Scour
Side Slope	3H:1V	3H:1V to 5H:1V	Sedimentation Hydraulics Geomorphology Armor Size & Scour
Bottom Elevation	-13.0 ft	-13.0 ft + 2ft Overdredge	None
Bottom Width	Varies	Varies, trivial change	None
Length	Approx. 620 ft	Approx. 620 ft	None
Waler Beam	Both sides of channel	Downstream side only	None
Turning Dolphins	Both sides of channel	Downstream side only	None
Turning Dolphins	5 ft O.D. w/ batter piles	4 ft O.D. monopile	Armor Size & Scour

2. Channel Sedimentation Study, Vol. 1

Analysis and estimates of sedimentation in the CER Volume 1 was conducted separately for the emergent and submerged parts of the launch channel.

Emergent channel sedimentation was estimated by assuming that sedimentation would occur due to settling (deposition) of all suspended sediment from the water column that enters the channel at each tidal cycle. By this method, sedimentation is proportional to the volume of water passing through the emergent channel and the suspended sediment concentration in this volume of water. Because channel width and suspended sediment concentration in the water column do not change with reconfiguration of the armor slopes, the amount of sedimentation in the emergent part of the channel should not change. It is our opinion that no revision to emergent channel sedimentation rates is required due to eliminating armor rock from part of the channel slopes.

Sedimentation in the submerged part of the channel was estimated assuming that sediment deposition would result from various sediment transport factors, including waves and cross-channel current fluxes. The combined effect (total sedimentation) from these factors was estimated using two different empirical methods. Each of these empirical methods was developed based on maintenance data from numerous navigation channels without the presence of armor rock on the channel slopes. Therefore, removing armor rock from the submerged part of the channel makes application of these methods and the results obtained from them more appropriate and reliable. No revision to estimates of sedimentation for a submerged part of the channel is required due to removal of armor rock.

In summary, rates and volumes of sedimentation and maintenance dredging requirements presented in the Volume 1 of the CER are applicable for the reconfigured the launch channel with reduced armor extent and modified channel side slopes. No change to the previous recommendations and conclusions is required.

3. Coastal Hydraulic and Geomorphologic Conditions, Vol. 2

Volume 2 of the CER documents existing hydraulic (tides, currents, and waves) and geomorphology (sediment transport, erosion/accumulation) conditions at the project site and presents possible changes of existing conditions due to construction and maintenance of the launch channel. The existing hydraulic and geomorphic conditions and possible changes to these conditions were determined using wave, current, and sediment transport numerical modeling. Based on previous experience with these models and practical knowledge of coastal conditions, removing the armor rock and changing the slope from 1V:3H to 1V:5H along a portion of the channel will not change hydraulic and geomorphic conclusions at the project site. Consider the two primary elements of launch channel reconfiguration: flattening of the slope and removal of armor rock.

- **Flattening of the slope:** With the more gradual 1V:5H slopes, the flow transition between the mudflats and launch channel will be less sudden. As a result, changes to hydraulics and geomorphology beyond the top of cut of the launch channel are expected to be even slighter for the reconfigured channel.

- **Removal of armor rock:** The absence of armor rock on the submerged side slopes would allow for less turbulent tidal and riverine flows in the lower water column at the unarmored launch channel side slopes. The flows would therefore be more similar to the existing condition. The difference between simulated flows with rock and without rock on side slope is likely less than the accuracy of the model.

4. Launch Channel Armor Size and Dolphin Scour Protection Design, Vol. 3

Volume 3 of the CER presents preliminary design recommendations for the minimum stable rock size for launch channel side slopes (1V:3H) under impact from propeller wash, determines depth of possible scour at turning dolphins and dolphins supporting the waler beams, and evaluates scour protection at the dolphins. No changes to recommendations of Volume 3 are required for the reconfigured channel.

- The initial armor size was developed for the fully armored initial channel configuration, for a maximum 75% thrust from the design tug (see Figure 4 in Vol. 3). This armor size has been carried forward for the reconfigured channel. The reconfigured launch channel armored slopes in the emergent channel would be exposed to approximately 25% thrust. Therefore, the recommended armor size is conservative for the reconfigured launch channel. No changes to the previous recommendations and conclusions are required.

- A slope flatter than 1V:3H in the reconfigured channel will result in slightly lower velocities on the slope, due to increased distance from the propeller to the rock. This may allow a slightly smaller stable size of rock for the reconfigured part of the slope. However, for consistency of the design and simplicity of construction we recommend the same type of material be placed on all armored slopes.

- The scour depth for the smaller 4 ft diameter monopile would be slightly less than for the initial 5 ft diameter pile. Therefore, previous recommendations on scour depth are more conservative. Design bottom velocities and rock size for scour protection remain the same. No change to the previous recommendations and conclusions is required.

5. References

CHE. 2011. *Technical Report, SR520 Coastal Engineering Report, Vol. 1 - 3.* Report in three volumes prepared for HNTB.

HNTB. 2011. *SR520 Pontoon Construction Design-Build Project, Package B - Site Facility Improvements.* Drawings provided by HNTB on 1/20/2011.

Technical Memorandum

SR520 Pontoon Construction Design-Build Project Launch Channel Armor Filter Layer Conformance with RFP

1. Introduction and Objective

Coast & Harbor Engineering (CHE) has prepared this technical memorandum to assist HNTB with responding on WSDOT Comment 2: "The launch channel riprap was designed and constructed without a filter layer, per RFP Section 2.13.2.1. Please provide documentation that demonstrates this design meets the standard contained in the *United States Army Corp of Engineers, EM 1110-2-1100: Coastal Engineering Manual, Parts I – IV and Appendix A, August 2008"*. This technical memorandum does not address the constructed condition of the channel armor, performance of the channel armor after construction, or current condition of the channel armor.

2. Design Summary

The launch channel armor was originally designed as a two-layer system with geotextile filter fabric (armor layer, bedding layer, geotextile filter fabric). The project geotechnical engineer, Shannon & Wilson, determined that such a design would result in an unacceptable safety factor for slope stability for the site soil conditions, and therefore, a second design was developed by CHE. This second design consisted of three layers: an armor layer, bedding layer, and filter layer. The armor stone was selected to remain stable under propeller-induced velocities, as documented in CHE (2011a). The armor layer, bedding layer, and filter layer gradation were selected and designed to be in conformance with the requirements in the CEM Part VI and taken to the RFC level.

Upon construction, Kiewit encountered very soft and wet soil conditions on the slope and requested, through RFI 4, that the filter layer be modified to provide a firm foundation layer for placement of the bedding and armor layers. CHE's analysis and recommendations on this matter is summarized in CHE (2011b) dated November 11, 2011. The final design consisted of the original armor layer (W_{15} = 150 lbs, W_{50} = 375 lbs), reduced thickness bedding layer (D_{85} = 5 inches, D_{15}=2 inches or 0.17 feet), and substitution of WSDOT standard light loose riprap (LLR) foundation layer for the filter layer. Materials and thicknesses are documented on the drawings dated November 11, 2011.

3. Conformance of the Design to Applicable CEM Standards

The CEM provides design requirements for the interface between cover layers and under layers in rubble mound structure design for wave action. Per WSDOT Comment 2 it is requested to provide documentation how the design criterion is met for "the riprap and bedding layer and the interface between the bedding layer and the subgrade soils). This criterion is given in Formula (VI-5-114) on page VI-5-125 of the United States Army Corp of Engineers, EM 1110-2-1100: Coastal Engineering Manual, Parts I – IV and Appendix A, August 2008 as follows: D15 (Cover) must be less than or equal to 5D85 (Under) (VI-5-114)"

First, consider the interface between the cover layer (armor layer D_{15}= 0.95 feet) and underlayer (bedding layer, D_{85} = 5 inches, 0.41 feet). Per equation VI-5-114 we compute, in agreement with the criterion:

$$0.95 \text{ ft} \leq 5 \times 0.41 \text{ ft} \qquad \text{(VI-5-114)}$$

Second, consider the interface between the cover layer (bedding layer D_{15}= 0.17 feet) and underlayer (LLR, D_{85} = 1.2 ft). Per equation VI-5-114 we compute, in agreement with the criterion:

$$0.17 \text{ ft} \leq 5 \times 1.2 \text{ ft} \qquad \text{(VI-5-114)}$$

As shown above, the EM 1110-2-1100 criteria for interface between the armor layer/bedding layer and bedding layer/light loose riprap layer has been satisfied by CHE's design.

Finally, let us consider the interface between the LLR layer and subgrade soils. This interface was designed based on a concept, as stated in CHE (2011b), that light loose riprap eliminates the need for a filter later. Therefore, the physical conditions (or setup) for design were changed. LLR was designed as a firm foundation layer that works the native subgrade into the LLR material. The formula and reference criterion (VI-5-114) was not applicable for the interface between the LLR and the subgrade soils. Because of design conditions, the method of analysis was changed also that corresponds the recommendation by the Coastal Engineering Manual Part I-2-1 as follows: "…engineering …need to be … flexible to changes in the local condition. Coastal engineers, managers, and planners need to be aware that … Analytical tools and procedures may be suitable for a particular setting but totally inappropriate for another."

At the same time, the design using LLR as a foundation layer does have basis and support in the CEM, particularly as it relates to soft soils and creating a uniform layer for placement of cover layers. The CEM describes the need, function, and design guidance for filter layers in various sections of Part VI. The following are excerpts from the CEM with CHE commentary in *italics* on how the design of the LLR layer conforms to the CEM guidance.

CEM Part VI-4-3
(6) Filter layers. Smaller stones are used for filter layers over the foundation soil or in drainage applications. Placement is usually by dumping. Selection of stone for a particular project depends on the purpose of the project, design loads, and local availability of suitable stone.

CHE's design meets the definition of this section. The selection of the light loose riprap layer was based upon the purpose of the project, which upon encountering soft wet soils *became to develop a suitable and uniform finished surface for placement of the overlying bedding and armor layers.*

(2)…Placing bedding material over soft and organic bottom materials should force the soft material outward toward the edges of the bedding layer. When finished, filter and bedding layers should be free of mounds and windrows and coverage should be complete.

CHE's design meets the requirement of this section. Producing a filter layer free of mounds was not possible with the original RFC filter layer material. Therefore, a courser and thicker gradation of light loose riprap was used to produce a firm and uniform foundation layer. The ability of light loose riprap to produce a firm and uniform foundation, free from mounds and windrows (as described in CHE 2011b) is confirmed by Kiewit's local site experience. Incorporating practical experience into the design, was specifically supported by CEM Part VI-3-7 "(b)…Practical knowledge and/or experience about how construction will proceed helps the engineer to evaluate the possibilities and modify the design to best accommodate construction needs."

CEM Part VI-5-3

(1) Filter layer functions. Filter layers are designed to achieve one or more of the following objectives in coastal structures:

- Filter functions can be achieved using either one or more layer of granular material or small stone of various grain sizes, …

CHE's design meets this definition. Light loose riprap is a small stone of various grain sizes.

- Prevent migration of underlying sand or soil particles through the filter layer voids into the overlying rubble-mound structure layers. Leeching of base material could be caused by turbulent flow within the structure or by excessive pore pressures that can wash out fine particles. Without a filter layer, foundation or underlayer material would be lost and the stones in the structure layer over the filter would sink into the void resulting in differential settlement and decreased structure crest elevation.

CHE's design meets this requirement. Based upon Kiewit's experience and encountered soil conditions, the light loose riprap was selected to stabilize the native soils, therefore minimizing differential settlement. The thickness of the foundation layer was increased to prevent the opportunity for soil particles to move through it. Placement of light loose riprap will change foundation conditions; thus, eliminating or reducing the need for a filtering layer.

- Distribution of structure weight. A bedding filter layer helps to distribute the structure's weight over the underlying base material to provide more uniform settlement…

Providing a firm foundation layer allowed for a level bedding layer to be accomplished.

- Reduction of hydrodynamic loads on the structure's outer stone layers. A granular filter layer can help dissipate flow energy whereas a geotextile filter will not be as effective in this regard.

Light loose riprap is a granular material and therefore helps to dissipate flow energy for the site conditions (propeller wash velocities).

4. Summary

Based on the analysis and discussion presented above, it is our opinion that the launch channel armor filter layer design meets the applicable standards contained in the *United States Army Corp of Engineers, EM 1110-2-1100: Coastal Engineering Manual, Parts I – VI and Appendix A, August 2008.*

5. References

CHE. 2011a. *Technical Report, SR520 Coastal Engineering Report, Vol. 1 - 3.* Report in three volumes prepared for HNTB.

CHE. 2011b. *Technical Memorandum, SR520 Pontoon Construction Design-Build Project RFI Proposed Changes to Channel Armor Sections.* Report prepared January 24, 2011.

From: Andrew.Kragt
Sent: Friday, September 13, 2013 1:19 PM
To: Nik.Schriener
Subject: FW: Geotechnical Action Items
Attachments: CHE Memo on Rip Rap.pdf; ATT00001.txt

From: Richard Kittler [mailto:RKITTLER@HNTB.com]
Sent: Tuesday, September 03, 2013 3:03 PM
To: Andrew.Kragt; Kevin.Tucker
Cc: Thomas Schnetzer; 'Bob Mitchell'; Vladimir Shepsis (vladimir@coastharboreng.com)
Subject: RE: Geotechnical Action Items

Andrew, Kevin,
We have a technical memo from Coast & Harbor Engineering providing additional information for WSDOT. Please review the following revised response to the WSDOT Question/Comment #2 and the attached memo, and let us know if you will need anything further to close out that comment.

HNTB revised response to Comment #2 (7/25/13) –

RFI 273 (submitted 11/9/11) and NDC 033 (submitted 11/16/11) maintained a three layer system. The bedding layer was reduced to 1 foot minimum thickness. The filter layer was increased to 2 feet minimum thickness and the gradation was revised to be the equivalent of light loose riprap. The heavier gradation of the filter layer provided a more consistent and constructible working surface for placing the bedding layer and provided slope stability for the channel cut slope. The D15 cover size of the armor layer, per the gradation on sheet LC4 of NDC 033, is 750 lb rock (approximately 23 inches dimension) and the D85 under size of the bedding layer is approximately 5 inches. 23 inches is less than 5xD85 = 25 inches, so it meets the interface gradation requirements. The D15 cover size of the bedding layer is approximately 2.5 inches and the D85 under size of the filter layer (Light Loose Riprap at 50 lb per WSDOT specs) is approximately 9 inches dimension. 2.5 inches is less than 5xD85 = 45 inches, so it meets the interface gradation requirements.

RFI 273 included both the geotechnical engineer's evaluation and the coastal engineer's evaluation. The gradations of the three layers meet all design requirements to; prohibit migration of material from the filter layer out through the bedding layer, prohibit migration of material from the bedding layer out through the armor layer, and to keep the existing excavated subgrade slope stabilized during and after construction.

The armor layer gradation specifications proposed on the NDC 033 revised slope protection sheet were reviewed and approved by the geotechnical engineer of record as an acceptable substitution for the generic "heavy rip rap" material recommended in the original geotechnical report for slope stability.

The Design Basis Manual in the O&M will be updated to reflect the revised launch channel section noted in RFI 273 and NDC 033.
The riprap section in front of the gate is not slope protection. It was placed in front of the gate as a flag to the dredging operations, should they get too close to the edge of the casting basin.

Regards,

Richard Kittler
Project Manager

HNTB Corporation
600 108th Avenue NE, Suite 900
Bellevue, WA 98004
(425) 455-3555
Direct (425) 450-2556
Fax (425) 453-9179
Email rkittler@hntb.com

From: Thomas Schnetzer
Sent: Thursday, August 15, 2013 6:12 AM
To: 'Andrew.Kragt@kiewit.com'
Cc: Richard Kittler; 'Bob Mitchell'
Subject: FW: Geotechnical Action Items

Andy,
Attached is the info we discussed yesterday.

A few additions to what's called out below:
Question 1 – the SR520 Site Surcharge is attached (was originally sent over on 6/24/13).
Question 2 – CHE has indicated they will have additional info for us next week.
Questions 3 & 4 – S&W were to send memo
Question 5 – memo attached (originally sent 6/25/13)
Question 6 – memo attached (originally sent 6/25/13)
PRV – memo attached (originally sent 6/25/13)
Sand on slope – memo attached (originally sent 6/25/13)

Bob, was the final memo for questions 3 & 4 sent? I didn't see it in my quick scan through the info this morning.

Call with questions.

Thanks,

Tom Schnetzer

HNTB Corporation
600 108th Ave. NE, Suite 900
Bellevue, WA 98004
Tel: 425.450.2576
Cell: 206.459.5624
Fax: 425.453.9179
tschnetzer@hntb.com

From: Thomas Schnetzer
Sent: Saturday, June 22, 2013 2:12 AM
To: David.Bowman@kiewit.com; cody.bishop@kiewit.com; 'Will.Morgan@kiewit.com'; Trevor Lighty; 'Bob Mitchell'; Richard Kittler
Subject: FW: Geotechnical Action Items

See below for status of geotechnical action items.
Green highlight below indicates material provided.
Yellow is material still being developed.

I am out of the office and will be out of contact until Thursday (then periodically available via email/cell phone).
Rick Kittler will be covering in my absence.

Thanks,

Tom Schnetzer

HNTB Corporation
600 108th Ave. NE, Suite 900
Bellevue, WA 98004
Tel: 425.450.2576
Cell: 206.459.5624
Fax: 425.453.9179
tschnetzer@hntb.com

From: Will.Morgan
Sent: Tuesday, June 11, 2013 3:41 PM
To: bob.mitchell@shanwil.com; tschnetzer@hntb.com
Cc: Cody.Bishop; David.Bowman; ReyesRV@wsdot.wa.gov
Subject: Geotechnical Action Items

Gentlemen,

After yesterday's meeting regarding the 6 Geotechnical Questions, I have compiled the following list of action items to hopefully close these issues out:

- Question 1: Shannon & Wilson and HNTB to create a site map with appropriate Surcharge Limits displayed for final O & M Manual. Site map to address anywhere on site a 250 psf load is allowed and/or areas with no load restrictions. – Load info received from S&W. Rick Kittler (HNTB) will be putting together the site map on Monday.
- Question 2: HNTB to coordinate with coastal engineer to ensure riprap and Shannon & Wilson drainage layer design standards have been met and that the information has been sent to K-G. HNTB was going to verify the contract design requirements that Bill referenced. Stamped Letter. See attached information that was included with RFI 273.
    - o CHE provided the memo noting that the armor layer and bedding layer are acceptable for the launch channel armoring, but noted that K-G would have to evaluate constructability (considered done since it's been constructed and because K-G has done

similar sections as noted in CHE's memo) and that a geotechnical engineer would have to confirm that the bedding layer will work as the filter layer.
    - S&W provided the memo noting that KG's proposed replacement material for the filter layer may not provide the functions of a filter layer. However, because of replacing soft underlayer material and creating a firm foundation with rip rap, it may reduce the need for a filtering layer as was specified previously.
- Question 3: Shannon & Wilson to compile piezometer data obtained from both float outs (before, during and after pontoon float out) to document basin groundwater levels and permanent ground water levels and confirm that this information falls in line with Shannon & Wilson's design assumptions. Provide stamped letter comparing assumptions to float out data collected as well as address any slope stability concerns in the South West corner of the basin. – Bob, please send info directly to group on this email when complete.
- Question 4: Combined with Question 3. No action.
- Question 5: Shannon & Wilson to submit slope cross section along with field log documentation and mapping of surface cracking and provide stamped letter confirming fix was done correctly and cracking from pump truck was a non-issue. Letter to address any residual strength in the slope, slope Factor of Safety meets the required 1.3 standard, and address that the surface cracking was not a failure. – Bob, please send info directly to group on this email when complete.
- Question 6: KG to provide Shannon & Wilson with settlement survey data. Shannon & Wilson to provide stamped letter to WSDOT specifying their findings. Will will work with Rafael to confirm if K-G met the contract requirements for submission geotech info and adherence to our plans. Change order or plan revisions may be necessary. – Has info been sent to Shannon & Wilson to date?
- Pressure Relief Valve (PRV): Shannon & Wilson to provide narrative discussing possible reasons for PRV "pop" and that the pressures are not going to impact the basin slab. Stamped Letter. Norma and Will to investigate WAC regulations on this matter. – Shannon & Wilson have estimated the uplift pressure to be 70 to 80 PSF in a small area. KPFF is looking at the effect up this uplift on the slab (don't expect it be an issue, but is doing a quick confirmation check) and expected to be done on 6/21/13. If the analysis indicates no issue, KPFF will send an email noting that the effects of the pressure result in a minimal change in stresses and appear acceptable. Bob, please send memo to all when complete.
- Fish Handling sand on side slopes: Shannon & Wilson to provide approval the use of sand on the side slopes. Stamped Letter. - Bob, please send info directly to group on this email when complete. My recollection was that this was a check for slope stability from the added sand only (WSDOT directed the addition of the sand).
- Cal Portland Paving: HNTB to produce a design package for the proposed paving and site work at the Cal Portland batch plant. – Drawing submitted as attachment. If it looks correct, we can issue as an NDC.

*HNTB and Shannon & Wilson will respond by 6/24/13.

Please let me know if you have any questions.

Thank You,

Kiewit
Infrastructure Group

Will Morgan
Engineer, SR 520 Pontoons

KIEWIT INFRASTRUCTURE WEST CO.
1301 West Heron Street, Aberdeen, WA 98520
(360) 689-2104 Cell
(360) 500-4457 Office
kiewit.com   Equal Opportunity Employer